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INVESTIGATION OF THE EFFECTIVENESS OF EXISTING GE DESIGN METHODOLOGY IN PROVIDING ADEQUATE ICTURAL RESISTANCE TO SEISMIC DISTURBANCES.

Phase II: Analytical Investigations of the Seismic Response of Long Multiple-Span Highway Bridges

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16. Abstract This report is the second in a series to result from the investigation, "An Investigation of the Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances", sponsored by the U. S. Department of Transportation, Federal Highway Administration. Descriptions are given to the analytical investigations of the seismic response of long, multiple-span, highway bridge structures of the type which suffered heavy damages during the San Fernando earthquake of February 9, 1971.

Linear and nonlinear mathematical modeling of this type of bridge structural system is presented. A three-dimensional elasto-plastic flexural column model suitable for modeling the coupled inelastic behavior of reinforced concrete bridge columns is described in detail. A nonlinear mathematical model for simulating the nonlinear discontinuous behavior of bridge expansion joints is also presented. Then, appropriate linear and nonlinear analytical procedures are described for determining the seismic response of this type of bridge structure.

Nonlinear seismic responses are presented for three prototype long, multiple-span, reinforced concrete highway overcrossing structures. Parameter studies carried out on these bridges are described, and the analytical results are presented, discussed, and correlated with the apparent prototype behavior observed during the San Fernando earthquake.

Finally, based on the analytical seismic responses presented, general conclusions and recommendations related to the fundamental seismic design methodology of long, multiple-span highway bridges are deduced and summarized.

This report is the second in a series. The others in the series are:

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PREFACE

The investigation with interpretation as described in this report was sponsored by the U. S. Department of Transportation, Federal Highway Administration, under Contract No. DOT-FH-11-7798 covering the period July 1, 1971 through September 30, 1974.

The general investigation called for in this contract is under the supervision and technical responsibility of Professors R. W. Clough, W. G. Godden, and J. Penzien. Professor Penzien acts as principal investigator.

ACKNOWLEDGEMENT

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I INTRODUCTION

In the past, numerous highway bridges have suffered extensive damages due to strong motion earthquakes [1,2]*. The older bridges, consisting of single or multiple simple truss or girder spans supported on massive piers and abutments, were particularly vulnerable to the action of strong ground motions. Seismic damages were most commonly caused by foundation failures resulting from excessive ground deformation and/or loss of stability and bearing capacity of the foundation soils. As a direct result, the sub-structures often tilted, settled, slid, or even overturned; thus severe cracking or complete failure was often experienced. These large support displacements also caused relative shifting of and damage to the super-structures, induced failures within the bearing supports, and even caused spans to fall off their supports. It is significant to note that very little damage occurred to these older structures as a direct result of structural vibration effects.

Certain types of modern highway bridges may, on the other hand, be quite susceptible to damage from strong ground vibration effects. This fact became very evident during the San Fernando earthquake of February 9, 1971, when numerous reinforced concrete highway bridges suffered severe damages [3,4]. These structures generally fall into two classifications (1) long, multiple-span bridges, and (2) short, single or multiple span bridges.

The long, multiple-span bridges usually consist of either straight or curved continuous reinforced concrete box girder decks supported on reinforced concrete columns and diaphragm abutments. The decks which are continuous over many spans are provided with intermediate expansion joints

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* Numbers in brackets refer to Bibliography numbers.

having restrainer bars tieing adjacent spans together. Although the overall bridge system in this case is an integrated space frame, significant structural discontinuities result from the presence of these joints. The lateral rigidity of the overall continuous deck system is therefore greatly reduced. Consequently, the lateral resistance of the total bridge structure is provided mainly by its supporting columns. These columns are often of various lengths consistent with the ground surface profile. Structure-foundation interaction effects may be relatively small for the taller bridges but can be quite large for the shorter bridges in which case they must be included in the analysis. In any case, the dynamic response of abutment backfills have negligible effect on the overall dynamic response of these bridges.

During a severe earthquake, long multiple-span bridge structures of this type can develop large amplitude oscillations. These oscillations may cause large cyclic inelastic deformations, of a coupled form, to develop in the columns. Also, cyclic slippage of the Coulomb type can take place in the expansion joints causing multiple impacts and separations to occur. These separations may be sufficiently large to cause yielding in the tension restrainer bars and, as experienced in San Fernando, may even cause deck spans to fall off their supports. While yielding of the restrainer bars can be tolerated, collapse of the bridge system is unacceptable.

From the above description, it is quite apparent that the dynamic response characteristics of long multiple-span reinforced concrete bridge structures are very complex due to the nonlinear, discontinuous behavior in the expansion joints under combined loading conditions. Because these characteristics are unique, one cannot compare the seismic response of this type of structure directly with the dynamic response of buildings under similar excitation conditions. Therefore, it is essential that the dynamic response and failure characteristics of these bridges be investigated separately by analytical and experimental means and that the results of these investigations be fully correlated with field evidence.

Turning our attention now to the second class of bridge structures mentioned above, i.e., short, single or multiple span bridges, it is immediately apparent that structure-foundation interaction effects

greatly influence the seismic response characteristics of this class of structure. Because they lack intermediate expansion joints, the deck responds essentially as a rigid body transferring its seismic loads through the supporting columns and abutments to the foundation.

The seismic response of abutment backfills may be in phase with the response of the bridge deck in such a manner that large dynamic active pressures develop which are additive to the seismic forces in the deck, thus greatly amplifying the dynamic response of the complete structural system. If the bridge should be skewed, as is often the case, the active forces on abutment can develop large twisting moments about a vertical axis through the elastic center of the bridge. These moments may become sufficiently large to cause failures of columns and abutments [1,2,3].

Obviously, any seismic analysis of this type of structure must consider foundation and abutment backfill soils as part of the complete structural system, if realistic results are to be obtained.

As previously mentioned, the 1971 San Fernando earthquake caused heavy damages to both classes of modern reinforced concrete bridges described above which were designed by the tradiational elastic approach, i.e., by the equivalent static seismic coefficient method similar to that previously adopted for buildings [5]. Because of this experience, it was immediately apparent that the seismic requirements used were inadequate and that the design methodology should be critically examined. Action was quickly taken following the earthquake to correct certain design deficiencies [6]; however, the basic static approach to design still remains in effect.

Recognizing the urgent need for both theoretical and experimental research related directly to seismic effects on bridge structures, a three-year investigation entitled "An Investigation Of The Effectiveness of Existing Bridge Design Methodology in Providing Adequate Structural Resistance to Seismic Disturbances" was initiated in 1971 within the Earthquake Engineering Research Center, University of California, Berkeley, under the sponsorship of the U. S. Department of Transportation, Federal Highway Administration. This investigation consists of the following

five phases:

(1) A thorough review of the world's literature on seismic effects on highway bridge structures including damages to bridges during the San Fernando earthquake of February 9, 1971.
(2) An analytical investigation of the dynamic response of long, multiple-span, highway overcrossings of the type which suffered heavy damages during the 1971 San Fernando earthquake.
(3) An analytical investigation of the dynamic response of short, single and multiple span, highway overcrossings of the type which suffered heavy damages during the 1971 San Fernando earthquake.
(4) Detailed model experiments on a shaking table to provide dynamic response data similar to prototype behavior which can be used to verify the validity of theoretical response predictions.
(5) A thorough comparison of dynamic response obtained from analyses, experiments, and field experience followed by the preparation of recommendations for changes in seismic design specifications and methodology as necessary to provide adequate protection against future earthquakes.

The final report covering Phase 1 has recently been published [1,2] and the present report is the final report covering Phase 2.

The primary objectives of the investigation in Phase 2 are to develop suitable analytical procedures for linear and nonlinear three-dimensional earthquake response analysis of long, multiple-span, modern highway bridges of the type which suffered heavy damages during the 1971 San Fernando earthquake and to identify and investigate the important parameters which significantly affect the response of this type of bridge system.

To achieve these objectives, linear and nonlinear mathematical models suitable for modelling the dynamic characteristics of this type of bridge have been defined and analytical procedures and computer

programs have been developed for determining the seismic response of the complete bridge system to arbitrary, but prescribed, earthquake excitations.

Linear and nonlinear seismic analyses have been performed on three typical major overcrossings using earthquake ground motions of several intensities. These bridges are (1) the South Connector Overcrossing of the Golden State freeway and Antelope Valley freeway interchange (5/14 South Connector Overcrossing), (2) the proposed curved Figueroa Street Undercrossing Connector in the Los Angeles area, and (3) a straightened version of the Figueroa Street Undercrossing Connector. Various parameter studies have been carried out for these bridges and the results are discussed and correlated with the apparent behavior of this type of structure during the San Fernando earthquake. In particular, the causes of collapse of the 5/14 South Connector Overcrossing are identified and examined.

Finally, based on the analytical results obtained, some general conclusions and recommendations related to fundamental seismic design methodology have been formulated.

Chapter 2 of this report describes the linear and nonlinear mathematical modelling of typical, long, multiple-span, modern highway overcrossings; Chapter 3 presents in detail a three-dimensional elasto-plastic flexural column model suitable for representing the coupled yielding behavior of columns under large cyclic deformations; Chapter 4 defines an appropriate nonlinear mathematical model for expansion faults which simulate the coupled effects of impact slippage with Coulomb friction and separation under the action of elasto-plastic restrainer bar forces; Chapter 5 and 6 describe the analytical procedures and computer programs; Chapter 7 presents the numerical results of the investigation; Chapter 8 gives a discussion of the numerical results, and Chapter 9 summarizes the conclusions and recommendations.

A. STRUCTURAL SYSTEM

The structural system of this type of bridge consists of a multiple-span continuous box girder deck supported on and rigidly connected to reinforced concrete columns and diaphragm abutments. The deck may be straight or curved in both horizontal and elevation views and is supported at discrete locations along its longitudinal axis by either centrally positioned columns or by transverse rows of multiple columns. Intermediate expansion joints divide the deck into several segments. Figure 1 shows the structural lay-out of the 5/14 South Connector Overcrossing which is typical of this type of bridge.

The entire structural system exhibits characteristics of a continuous space frame. Its dynamic response to earthquake excitations is of lower mode type; hence, a mathematical model of discrete form can be used to approximate the continuous system. This form of modelling leads to a system having a finite number of degrees of freedom.

Following the standard finite element procedure, these degrees of freedom are chosen as the nodal displacements of the discrete finite element model. For a three dimensional model, each nodal point usually has 6 degrees of freedom, i.e., 3 translation components and 3 rotation components. Internal constraints may however reduce this number at some nodal points.

For purposes of dynamic analysis, the stiffness, mass, and damping properties of each finite element must be realistically defined and the characteristics of each expansion joint must be properly modelled. These features of the overall system are described in detail in the subsequent sections of this report.

B. STIFFNESS IDEALIZATION

The finite element idealization of the complete bridge system

results in a stiffness matrix which is an assemblage of the generalized stiffness matrices for individual elements, i.e., in symbolic form:

$$\underline{K} = \sum_{i=1}^{N} \underline{k}_i \tag{1}$$

where $\underline{K}$ is the total stiffness matrix for the entire bridge, $\underline{k}_i$ is the stiffness matrix for element i, and N is the total number of elements in the system.

For small amplitude response, the bridge system may be modelled by a set of linear elastic elements; however, when subjected to high amplitude response as occurs during severe earthquakes, certain critical regions of the structure may undergo large cyclic inelastic deformations. Therefore, nonlinear finite elements must be chosen for the mathematical model which have realistic nonlinear hysteretic force-deformation characteristics. The stiffnesses of these elements are time dependent and are functions of element deformations and deformation histories. Usually, they are linearized for analysis in a piecewise fashion using tangent stiffnesses at discrete times. Thus, the total stiffness matrix for the entire structure may be written in the symbolic form:

$$\underline{K}_t = \sum_{i=1}^{N} \underline{k}_{ti} \tag{2}$$

where $\underline{K}_t$ is the total stiffness matrix at time t and $\underline{k}_{ti}$ is the stiffness matrix for element i at time t. It should be noted that all nonlinear elements referred to in this report account for material nonlinearities only. Nonlinearities arising from large geometry changes are not included as they are negligible for the levels of response considered acceptable in these structural systems.

1. Decks - The decks of most modern reinforced concrete highway bridges are of the curved box girder type as shown in Fig. 2a. The performance of these girders under static loadings has been investigated extensively [7 - 10]. The analytical methods employed used various types of elements for the deck structure such as simple

beam elements, folded plate elements, and quadralateral plate elements [7]. It has been shown that when determining internal stress distributions in constituent flanges and webs of box girders under localized loadings that rather elaborate methods of analysis, such as finite segment analysis or finite element analysis, must be used. However, when the external loadings are rather uniformly distributed and when only resultant forces on transverse cross-sections, i.e., 3 components of force and 3 components of moment, are required, a simple beam analysis is usually sufficient to yield accurate results. Thus, the mathematical model used herein for seismic analysis represents the deck by a series of either straight or curved beam elements as shown in Fig. 2b.

The stiffness matrix of a linear elastic straight beam element is found in most modern textbooks on matrix structural analysis [11] and the stiffness matrix of a linearly elastic, circularly curved, beam element can be found in papers by Morris [12] and Tezcan, et.al. [13].

Since a typical box girder deck is extremely stiff and strong in comparison with its supporting columns and abutments, the high amplitude bridge response produced during severe ground shaking will be caused primarily by deformations in the columns, abutments, and expansion joints. The deck will remain elastic and, therefore, can be modelled by linear elastic elements. Nonlinear yielding elements must, however, be used for columns, abutments, and the tension restrainer tie bars used in expansion joints.

2. Columns and Abutments - The structural behavior of columns can be adequately modelled using simple beam elements; see Fig. 2c. Because of large amplitude response, coupled inelastic flexural deformations may occur in these members. Therefore, nonlinear beam elements which realistically characterize the inelastic hysteretic behavior must be used. An appropriate elasto-plastic element under combined loading has been defined for this purpose and described in detail in Chapter 3.

The abutments of this type of bridge are usually of the diaphragm

type which act in the manner of shear wall elements. Failures are likely to be of the shear type causing excessive damage. Therefore, such failures should be avoided through proper design. Consequently, elastic behavior is always assumed for abutments in the present analysis.

3. Foundations - Complex mathematical models have been used for structure foundations in previous investigations [14, 15, 16]. However, a simpler and more approximate model has been adopted herein. This model consisting of 3 translational and 3 rotational springs is used to connect the base of each column and abutment to a rigid foundation where the seismic excitation is fully prescribed; see Fig. 2c. For linear analysis, the stiffnesses of these springs can be evaluated using linear elastic half-space theory similar to that used in the reference [15], or methods reported in the literature [17].

For large amplitude response, the foundation soils may undergo inelastic deformations of the hysteretic type. In this case, the six foundation springs should be bilinear hysteretic springs. The elastic stiffnesses of these springs can be obtained using standard methods; however, their yielding stiffnesses and yield values can be established only through extensive experimental studies on the dynamic properties of the foundation soils [15].

A very simple method, which is often used for modelling foundation flexibility, is to assume that the columns and abutments extend below the ground surface to specified depths at which fully flexity conditions are believed to have been reached. The ground excitations are then fully prescribed at these effective depths.

C. MASS IDEALIZATION

The continuous mass of the bridge structural system is modelled in discrete form by lumping element masses at their end nodal points. Since inertia forces are associated with each of the six degrees of freedom at a nodal point, each lumped mass must be assigned appropriate moments of inertia about its own three coordinate axes (x,y,z). It should also be noted that when conducting nonlinear dynamic analyses, the

instantaneous stiffness matrix may become singular in which case it is essential that mass moments of inertia be assigned to each rotational degree of freedom. Following this procedure, a diagonal mass matrix $\underline{m}_i$ is established for each element i (i = 1,2, ---,N). The diagonal mass matrix for the complete bridge system can then be assembled and expressed in the symbolic form:

$$\underline{M} = \sum_{i=1}^{N} \underline{m}_i \tag{3}$$

In determining overall dynamic response, this lumped mass method has been found to be quite adequate for analysis purposes [18].

D. DAMPING IDEALIZATION

Velocity dependent damping in the structural system is represented by a generalized damping matrix associated with the finite degrees of freedom permitted in the mathematical model. This matrix can be derived by consistent procedures similar to those used in deriving the stiffness matrix provided the internal damping mechanisms within each element are specified. The structural damping matrix for the complete structural system would then be of the symbolic form:

$$\underline{C} = \sum_{i=1}^{N} \underline{c}_i \tag{4}$$

where $\underline{c}_i$ is the damping matrix for the ith element.

In practice, however, it is very difficult to establish the basic characteristics of damping in the individual elements. Therefore, it is customary to assume that the damping forces of each element consists of one set which are proportional to the velocities of each mass point and a set which are proportional to the rates of deformation in the element [18]. Thus, the element damping matrix can be represented as:

$$\underline{c}_i = \alpha_i \, \underline{m}_i + \beta_i \, \underline{k}_i \tag{5}$$

where α_i and β_i are the scalar proportionality constants for the ith element. If the entire structure is made of the same basic material, it is reasonable to assume similar damping characteristics for each element in the system, i.e., α_i and β_i are independent of i in which case, combining Eqs. (4) and (5), the structural damping matrix becomes:

$$\underline{C} = \alpha \underline{M} + \beta \underline{K} \tag{6}$$

where

$$\alpha = \alpha_i \; ; \; \beta = \beta_i \qquad (i=1,2,---,N) \tag{7}$$

In the case of linear response, it can be shown that the two generalized parameters α and β can be determined after assigning damping ratios to the first two natural modes of vibration [19].

For nonlinear dynamic analysis, the viscous damping properties of the structure are much more difficult to access. However, it is reasonable to make assumptions similar to the elastic case, i.e., to assert one set of damping forces proportional to the nodal velocities, as assumed in the elastic case, and a second set which are proportional to the rates of elastic deformation only. In this case, the structural damping matrix takes the form:

$$\underline{C}_t = \alpha \underline{M} + \beta \underline{K}_t \tag{8}$$

where α and β are the constants given in Eq. (7).

E. EXPANSION JOINT IDEALIZATION

A typical expansion joint is shown in Figs. 3a and 3b. The bridge deck on one side of the expansion joint rests on a ledge of the deck on the opposite side. A number of transverse shear keys exist on the support ledge which prevent relative transverse motion of the deck within the joint. Longitudinal restrainer bars are attached across the expansion joint tieing together the end diaphragms of the adjacent deck spans. These tie bars are placed in such a way that they carry only tensile forces after a small tie gap is closed by joint separation. Another

small gap is provided in the joint to allow for thermal expansion of the deck spans. Vertical restrainers are installed in the expansion joint to prevent lifting of one span off the other along the support ledge.

Referring to Figs. 3a and 3b, the expansion joint characteristics may be summarized as follows:

(1) Relative translation of the deck spans along the longitudinal x axis cause impacts when the gap between adjacent spans is closed, slippage on the support ledge of the Coulomb type, and extensions of the longitudinal restrainer bars when joint separations exceed tie bar gaps.

(2) Relative translation of the spans along the transverse y axis is prevented by the shear keys.

(3) Relative translation of the spans along the vertical z axis is constrained by vertical restrainer bars.

(4) Relative rotation of the spans about the x axis is constrained by the vertical restrainer bars.

(5) Relative rotation of the spans about the y axis is freely permitted.

(6) Relative rotation of the spans about the z axis is coupled to the relative translation of spans along the x axis and may be partially or fully constrained by a combination of closure of the gap between decks, actions of the longitudinal restrainer bars, and the presence of Coulomb friction.

Because of these characteristics, the expansion joint behaves as a nonlinear discontinuous element causing different degrees of constraint during the motion of the structure. Therefore, the total number of degrees of freedom in the complete system changes from time to time during the response causing complexities to arise in the nonlinear dynamic analysis procedures.

Two limiting cases of expansion joint behavior, providing minimum and maximum constraint, are assumed for linear dynamic analysis. These two

linear cases are achieved by neglecting the effects of impact and longitudinal restrainer bar constraints and by assuming either zero or infinite Coulomb friction. The infinite friction case prevents relative translation of deck spans along the longitudinal x axis and prevents relative rotation about the z axis; thus, it characterizes the prototype dynamic behavior for low amplitude response. The zero friction case, on the other hand, approximates prototype behavior for high amplitude response following failure of the short longitudinal restrainer bars but prior to yielding in the columns.

The overall bridge structure is very stiff in the infinite friction case due to arching action of the deck from abutment to abutment which carries most of the lateral loads. It is however, relatively flexible in the zero friction case since arching action cannot occur and the lateral loads must be carried in the supporting columns.

F. LINEAR AND NONLINEAR MATHEMATICAL MODELS

In summary, the mathematical model used for linear dynamic analysis consists of (1) linear elastic straight beam elements, (2) linear elastic curved (circular) beam elements, (3) linear boundary spring elements, and (4) linear expansion joint elements, while the model used for nonlinear analysis consists of (1) linear elastic straight beam elements, (2) linear elastic curved beam elements, (3) elasto-plastic flexural beam elements, (4) bilinear hysteretic boundary spring elements, and (5) nonlinear expansion joint elements. Figure 4 shows the discrete parameter model for the 5/14 South Connector Overcrossing.

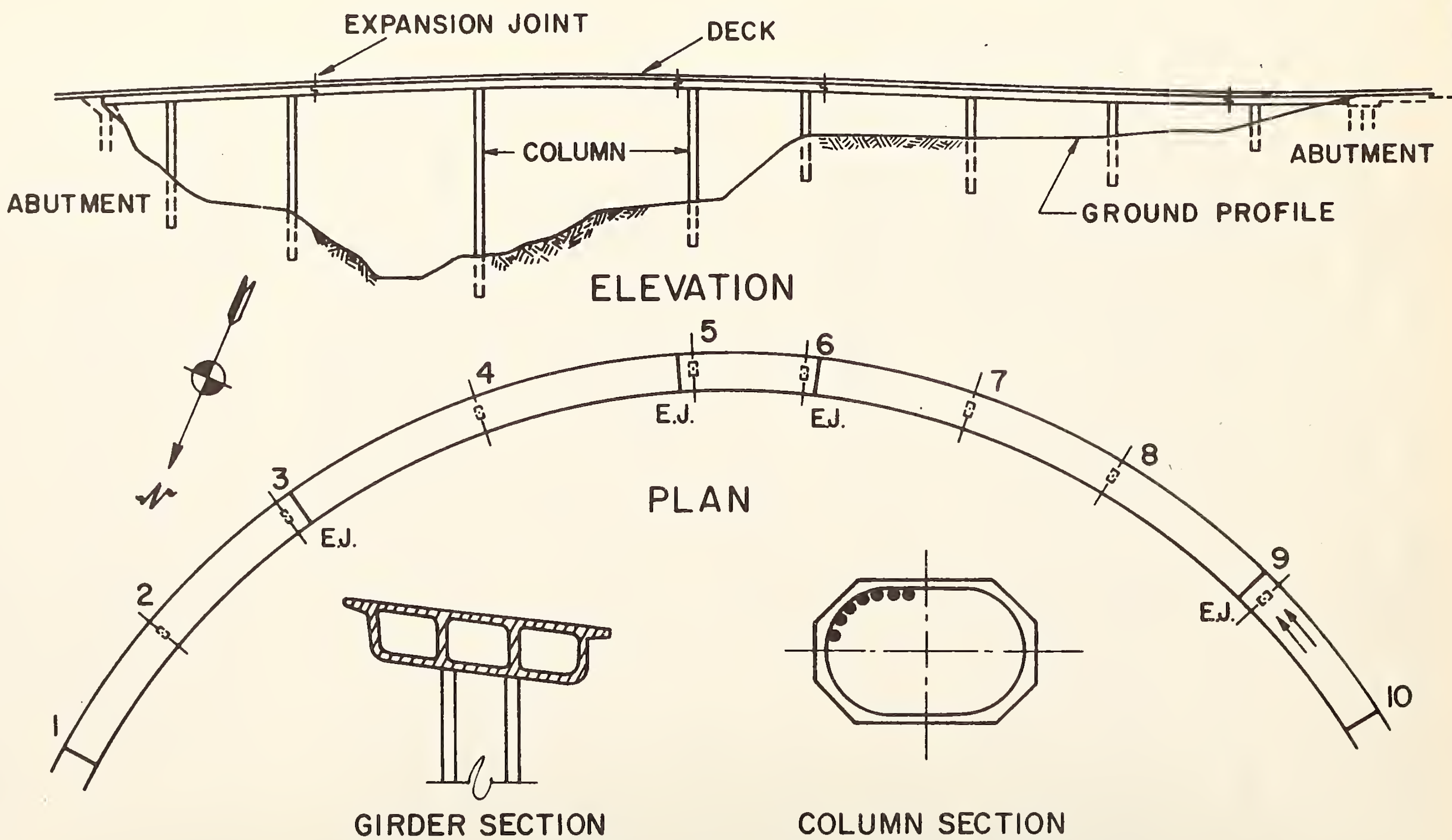


Fig. 1 5/14 South Connector Overcrossing

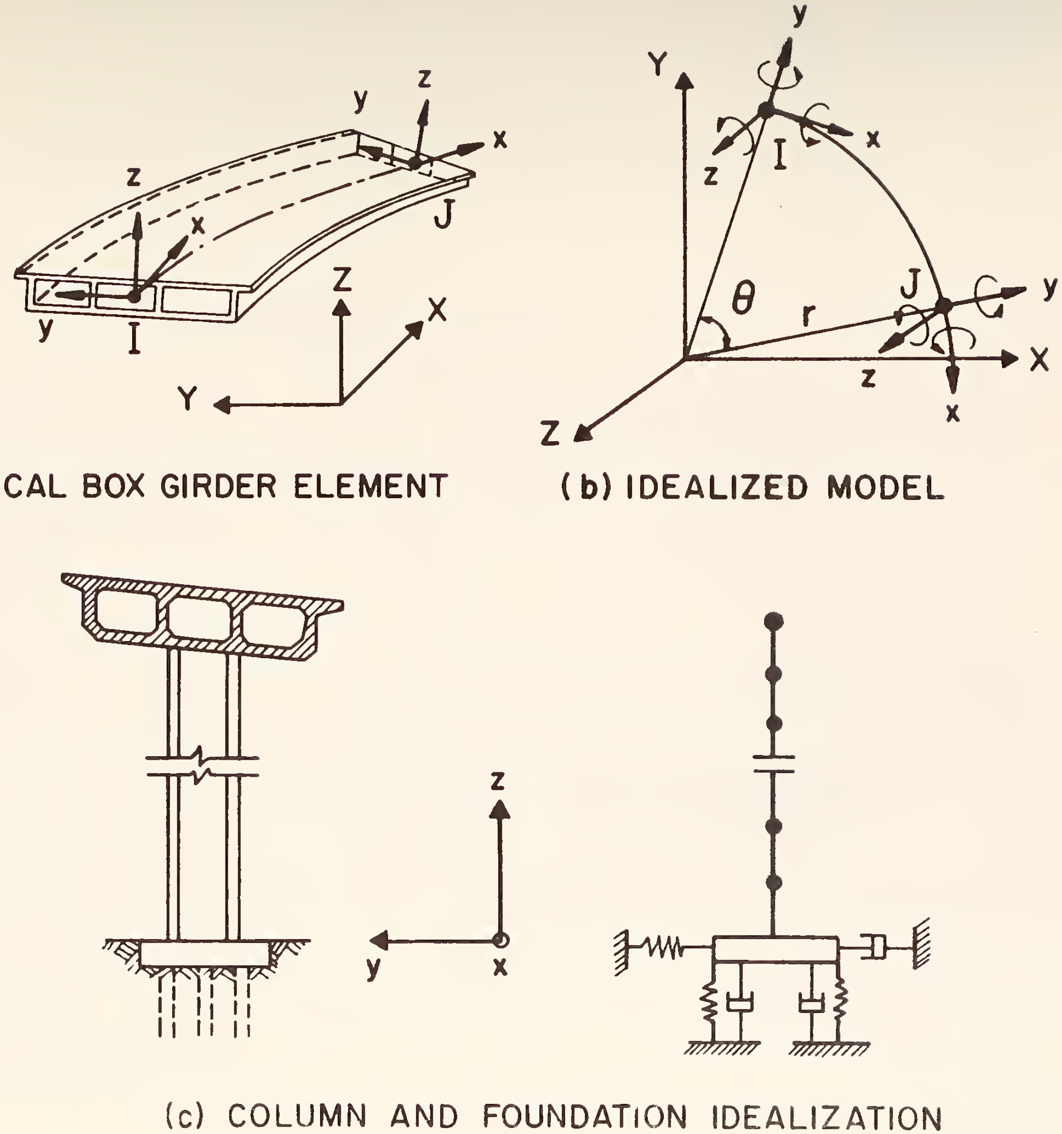


Fig. 2 Mathematical Modeling of Bridge Structural System

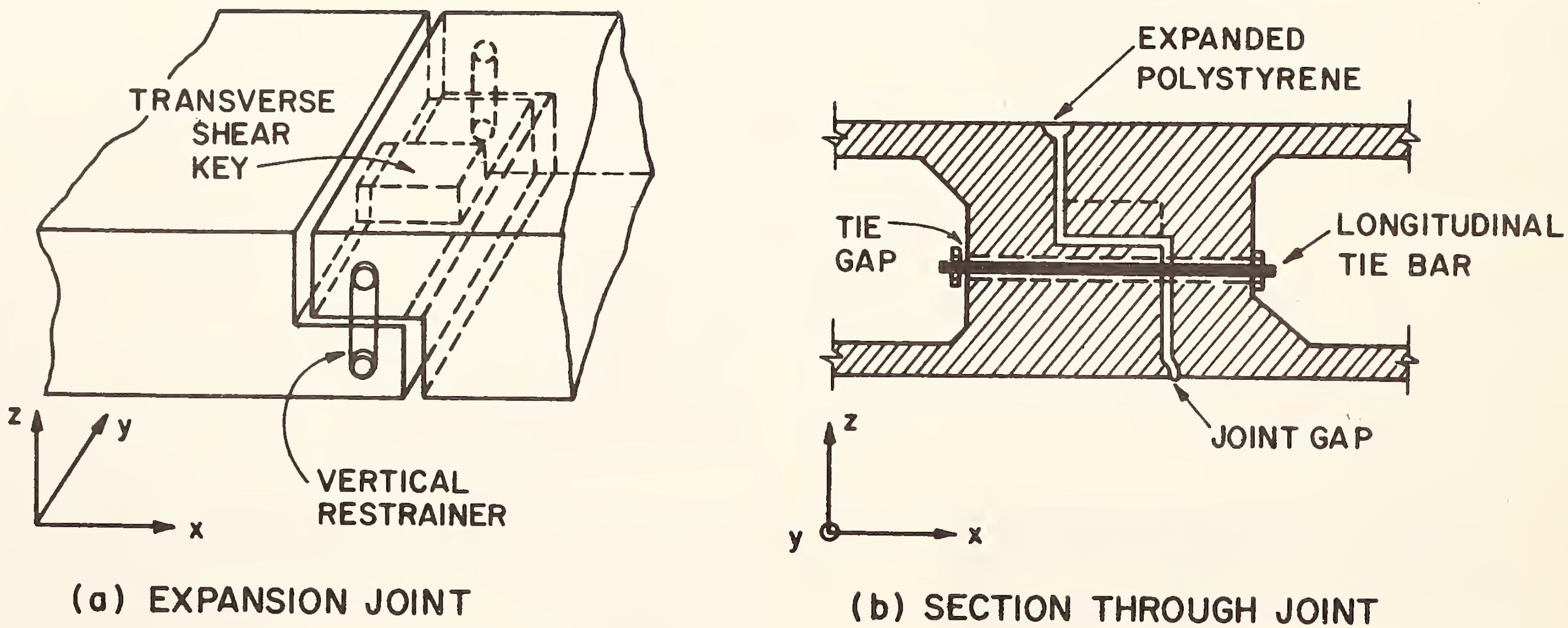


Fig. 3 Bridge Expansion Joint

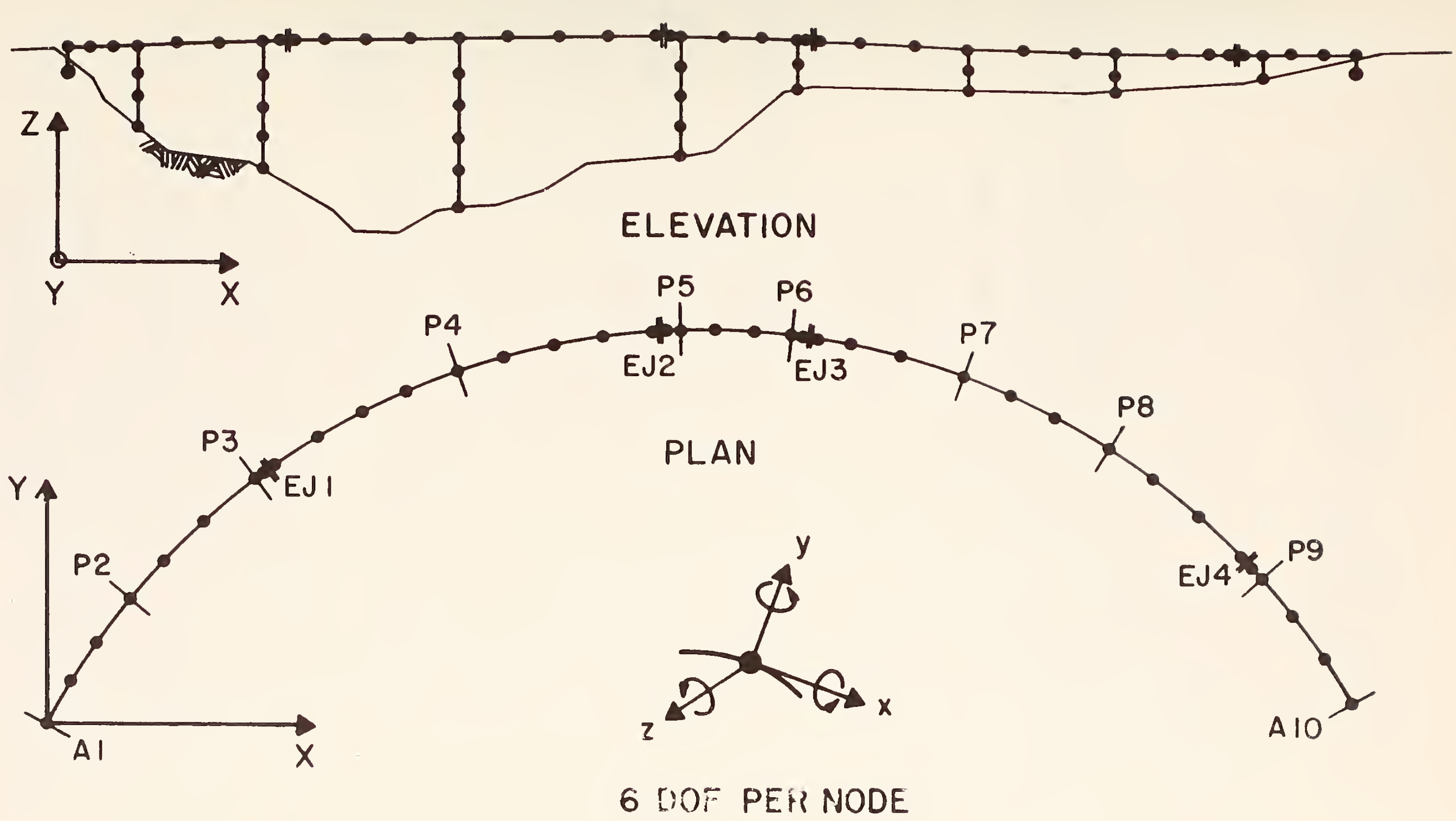


Fig. 4 Idealized Lumped Parameter System For 5/14 South Connector Overcrossing

III ELASTO-PLASTIC FLEXURAL COLUMN ELEMENT FOR REINFORCED CONCRETE COLUMNS

A. BASIC ASSUMPTIONS

As pointed out previously, limited inelastic deformations may be permitted in reinforced concrete bridge columns under severe earthquake conditions. These deformations should however be permitted only when adequate ductility and energy absorption capacities are available. Therefore, cyclic yielding should be allowed in flexure only where high ductility can be provided. Since shear inelastic deformations are usually brittle in character, they should be avoided. Thus, it is essential in providing high seismic resistance under severe overload conditions that reinforced concrete columns be sized and detailed so that yielding will be limited to flexure.

Because reinforced concrete is a highly heterogeneous composite material, the overall inelastic behavior of column elements should be defined in terms of generalized forces and their corresponding generalized deformations. Six generalized forces exist at any section of an element, namely, an axial force P, two components of transverse shear Q_y and Q_z, torsion T, and bending moments M_y and M_z, Fig. 5. Axial normal stresses are caused by generalized forces P, M_y, and M_z while transverse shear stresses are caused by Q_y, Q_z, and T. In constructing an inelastic element for reinforced concrete columns, the following basic assumptions have been made (see Fig. 5):

(1) The generalized force-deformation relations of the element follows that of an elasto-perfectly plastic (elasto-plastic) material model having yield strengths corresponding to the ultimate capacities of the member.

(2) The ultimate shear strengths, Q_{yu} and Q_{zu}, and the ultimate torsional strength, T_u, are infinitely large.

(3) The ultimate axial strength P_u and the ultimate bending strengths M_{yu} and M_{zu} are determined from the axial stress distribution present on the cross-section under ultimate conditions and are independent of the shear stresses caused by Q_y, Q_z, and T.

(4) The interaction among P_u, M_{yu}, and M_{zu} can be represented by a three-dimensional generalized yield surface in the force space spanned by P, M_y, and M_z.

(5) The element retains linear elastic behavior between nodal points; however, elasto-plastic behavior as defined in assumptions (1) through (4) above is permitted to occur over the cross-section at each end for a distance Δx approaching zero, i.e., the inelastic behavior is fully concentrated at the ends of the element.

Assumptions (1) through (4) relate to material properties within the concentrated elasto-plastic hinges at the ends of the elements. Between these hinges the material remains elastic. It should be understood that the above assumptions have been made to approximate the actual case in which inelastic deformations may occur at any section along a column. If a sufficient number of elements are used, a good approximation results. This is due to the fact that the real case is approached in the limit with increasing numbers of elements.

The inelastic column element resulting from the above assumptions may be constructed mathematically by carrying out the following three steps:

(1) Determine the ultimate strengths P_u, M_{yu}, and M_{zu} for the given section under ultimate conditions.

(2) Define the generalized yield function which controls the yielding interaction of P_u, M_{yu}, and M_{zu}.

(3) Formulate the elasto-plastic tangent stiffnesses in terms of the element force and corresponding nodal displacement increments during yielding.

The analytical details used in carrying out these steps, as well as the computational techniques used in the step-by-step dynamic analysis procedure, are presented in the subsequent sections of this chapter.

B. ULTIMATE STRENGTHS

The ultimate strengths of a reinforced concrete column subjected to combined axial load and biaxial bending can be determined using the following idealizations [20,21]:

(1) Plane sections remain plane.

(2) Concrete carries no tension.

(3) Maximum compressive strain of concrete equals 0.003.

(4) The concrete stress distribution in the compression zone can be replaced by an equivalent stress block having uniform intensity equal to 0.85 times the concrete cylinder strength f'_c and distributed over an equivalent compression zone bounded by the boundary of the cross-section and a straight line located at a distance $(1-k_1)c$ from and parallel to the neutral axis where c is the distance from the location of maximum compressive strain to the neutral axis and where

$$k_1 = \begin{cases} 0.85 & , \quad f'_c \leq 4000 \text{ psi} \\ 0.85-0.05\left(\dfrac{f'_c-4000}{1000}\right) & , \quad f'_c > 4000 \text{ psi} \end{cases} \tag{9}$$

(5) The stress-strain relation for the steel reinforcement follows an elasto-perfectly plastic law having equal yield stresses in tension and compression.

The idealized stress-strain distribution at ultimate condition is shown in Fig. 6 for a typical bridge column section. In this figure, the y axis is selected as the minor axis of the section and the z axis is selected as the major axis. Using conditions of equilibrium, the ultimate axial force P_u and the ultimate bending moments M_{yu} and M_{zu} can be expressed in the form

$$
\begin{aligned}
P_u &= -0.85\, f'_c A_c - \Sigma_c A_s f_{sc} + \Sigma_t A_s f_{st} \\
M_{yu} &= -0.85\, f'_c A_c z_c - \Sigma_c A_s f_{sc} z_{sc} + \Sigma_t A_s f_{st} z_{st} \qquad (10) \\
M_{zu} &= 0.85\, f'_c A_c y_c + \Sigma_c A_s f_{sc} y_{sc} - \Sigma_t A_s f_{st} y_{st}
\end{aligned}
$$

where the signs of P_u, M_{yu}, and M_{zu} agree with the convention shown in Fig. 6, the symbol Σ_c sums over the steel reinforcements in the compression zone, and Σ_t sums over the steel reinforcements in the tension zone.

Since the ultimate strengths P_u, M_{yu}, and M_{zu} of a given column section are nonlinear functions of the geometric parameters of the section, an iterative procedure must be used to compute their values. An initial position of the neutral axis is first assumed in this procedure; then, following an equilibrium correction using Eqs. (10), a new neutral axis position is determined. This procedure is repeated until convergence to the correct neutral axis position is sufficiently obtained.

Because modern bridge columns usually have rather complex shapes of cross-sections and the steel reinforcements are quite dense and peripherally arranged, the above described iterative procedure can be very tedious if carried out by traditional methods. For this reason, an efficient procedure using Newton's method was developed which can treat cross-sections of arbitrary shapes having arbitrary distributions of steel reinforcements. A computer program YIELD has been developed

for this purpose

Having calculated the numerical values of P_u, M_{yu}, and M_{zu} for a given cross-section and distribution of steel reinforcements using Eqs. (10), they represent a point in a three-dimensional generalized force space spanned by P, M_y, and M_z. If all such possible points were computed, they would define a three-dimensional surface which controls the interactions among ultimate strengths P_u, M_{yu}, and M_{zu}. It is convenient to represent this surface by two parameters, namely, an axial force ratio p_u and an eccentricity angle θ as defined by

$$p_u = \frac{P_u}{P_o}$$

$$\theta = \tan^{-1}\left(\frac{M_{yu}}{M_{zu}}\right) \tag{11}$$

where P_o is the ultimate axial compressive strength under conditions of no bending. The interaction surface can then be represented in terms of

$$\begin{aligned} P_u &= P_u\,(p_u,\theta) \quad ; \quad P_o = P_u\,(-1,0) \\ M_{yu} &= M_{yu}\,(p_u,\theta) \quad ; \quad M_{yo} = M_{yu}\,(0,0) \\ M_{zu} &= M_{zu}\,(p_u,\theta) \quad ; \quad M_{zo} = M_{zu}\,(0,\tfrac{\pi}{2}) \end{aligned} \tag{12}$$

For a fixed value of p_u, Eqs. (12) define the biaxial bending interaction curve between moments M_{yu} and M_{zu} and, for a fixed value of θ, these equations specify the interaction curve between the ultimate axial force P_u and the ultimate bending moment $M_u = (M_{yu}^{\ 2} + M_{zu}^{\ 2})^{\frac{1}{2}}$ about an axis located at an angle θ from the y axis. As an example, consider the typical bridge column section shown in Fig. 7. Ultimate strengths have been calculated for this section and interaction curves have been established. Figure 8 presents a single biaxial bending interaction curve which as shown is representative for widely differing values of p_u. The ultimate moments M_{yu} and M_{zu} in this case,

as in others, have been normalized with respect to M_{yo} and M_{zo}, respectively. Figure 9 shows a single interaction curve for axial force P_u and the normalized M_{yu} moment which is representative for θ over the full range $0 < \theta < 90°$.

The shape of the yield surface as represented by Eq. (10) is obviously dependent upon the geometric shape of the cross-section and upon the amount and distribution of reinforcing steel. It is very tedious however to generate the complete yield surface for each type of section under consideration. Therefore, for analytical purposes, approximate yield functions have been developed which depend on fewer parameters.

C. GENERALIZED YIELD FUNCTION

Many approximate yield surfaces have been proposed for reinforced concrete sections under combined loadings [22,23]. Most forms developed however are for rectangular sections having simple arrangements of reinforcing bars. Therefore, they cannot be readily applied to the more complex cases arising in modern bridge design.

The approximate yield surface developed herein follows a form similar to that proposed by Bresler [22] for biaxial bending and uses a cubic polynomial approximation for the axial force and bending interaction curves. As proposed by Bresler, the biaxial bending interaction curves for fixed values of p_u can be approximated by the relation

$$\left|\frac{M_{yu}}{M_{yp}}\right|^a + \left|\frac{M_{zu}}{M_{zp}}\right|^b = 1 \tag{13}$$

where a and b are constants which depend upon the cross-section geometries and where M_{yp} and M_{zp} are the ultimate bending moments about the y and z axes, respectively, for a fixed value of p_u and for M_{zu} and M_{yu} set equal to zero, respectively. Moments M_{yp} and M_{zp} can be approximated by the following cubic equations:

$$\left|\frac{M_{yp}}{M_{yo}}\right| = 1.0 + a_1 \left(\frac{P_u}{P_o}\right) + a_2 \left(\frac{P_u}{P_o}\right)^2 + a_3 \left(\frac{P_u}{P_o}\right)^3 \quad , \quad -P_o<P_u<P_t$$

$$\left|\frac{M_{zp}}{M_{zo}}\right| = 1.0 + b_1 \left(\frac{P_u}{P_o}\right) + b_2 \left(\frac{P_u}{P_o}\right)^2 + b_3 \left(\frac{P_u}{P_o}\right)^3 \quad , \quad -P_o<P_u<P_t \tag{14}$$

where P_t is the ultimate axial tension for the column under conditions of no bending and a_1, a_2, a_3, b_1, b_2, and b_3 are constants. Combining Eqs. (13) and (14), the approximate yield surface can be represented in the normalized form

$$\left|\frac{m_{yu}}{1 + a_1 p_u + a_2 p_u^2 + a_3 p_u^3}\right|^a + \left|\frac{m_{zu}}{1 + b_1 p_u + b_2 p_u^2 + b_3 p_u^3}\right|^b = 1 \tag{15}$$

or symbolically

$$f(p_u, m_{yu}, m_{zu}) = 1 \tag{16}$$

where

$$p_u = P_u/P_o, \quad m_{yu} = M_{yu}/M_{yo}, \quad m_{zu} = M_{zu}/M_{zo}$$

are the normalized resultant force components.

The normalized yield function, Eq. (16), contains eleven constants, namely, P_o, M_{yo}, M_{zo}, a, a_1, a_2, a_3, b, b_1, b_2, and b_3. These constants are determined by fitting smooth curves through ultimate strength values obtained by the methods previously described. For typical bridge columns which have nearly oval-shaped cross-sections and are peripherally reinforced, an ellipse can be used to approximate the biaxial bending interaction curve. In this case, a value of 2 is used for both a and b. If, on the other hand, the cross-section is a long, narrow, rectangular section, the corresponding interaction curve approaches a straight line in which case a and b both equal unity. Figure 10 shows a three-dimensional plot of the yield surface given by Eq. (15) which is smooth everywhere except

at $P = P_o$ and $P = P_t$ when a and b are both equal to 2.

In accordance with the definition of a yield function, if the values of P, M_y, and M_z are such that $f(p, m_y, m_z) < 1$, where $p = P/P_o$, $m_y = M_y/M_{yo}$, and $m_z = M_z/M_{zo}$, then the cross-section is in an elastic state. If $p = P_u$, $m_y = m_{yu}$, and $m_z = m_{zu}$, the section is in a yield (or impending yield) state. When $df = o$, the values of p_u, m_{yu}, and m_{zu} can change but they must always be compatible with the yield surface. If $df < o$, the section is being unloaded. A set of values for p, m_y, and m_z which gives $f(p, m_y, m_z) > 1$ is, of course, not possible using an elasto-perfectly plastic model.

D. TANGENT STIFFNESSES FOR ELASTO-PLASTIC FLEXURAL COLUMN ELEMENTS

In this section, the elasto-plastic tangent stiffness matrix $\underline{k}_t^{EP}$ which relates the element force increments $d\underline{S}$ to the element nodal displacement increments $d\underline{r}$ for an elasto-plastic flexural column element yielding at one or both ends will be derived using the generalized yield function $f(p_u, m_{yu}, m_{zu})$ established previously and the associated flow rule for an elasto-plastic solid. The desired relation is

$$d\underline{S} = \underline{k}_t^{EP}\, d\underline{r} \tag{17}$$

where

$$d\underline{S} = \begin{Bmatrix} d\underline{S}_I \\ \\ d\underline{S}_J \end{Bmatrix} \qquad d\underline{r} = \begin{Bmatrix} d\underline{r}_I \\ \\ d\underline{r}_J \end{Bmatrix} \tag{18}$$

where $d\underline{S}_K$, $d\underline{r}_K$, K = I, J are the vectors of element force increments and corresponding element nodal displacement increments at node K.

By virtue of assumption (5) in Section A and the assumption that the element deformation increments can be additively decomposed into an elastic component and a plastic component, the element nodal displacement increments $d\underline{r}$ can be expressed as

$$d\underline{r} = d\underline{r}^E + d\underline{r}^P \tag{19}$$

where

$$d\underline{r}^E = \begin{Bmatrix} d\underline{r}_I^E \\ \\ d\underline{r}_J^E \end{Bmatrix} \qquad d\underline{r}^P = \begin{Bmatrix} d\underline{r}_I^P \\ \\ d\underline{r}_J^P \end{Bmatrix} \tag{20}$$

and where $d\underline{r}_K^E$, K = I, J, represents the vector of elastic nodal displacement increments at node K, and $d\underline{r}_K^P$, K = I, J represents the plastic displacement increments resulting from concentrated plastic deformations at end K. The element forces are uniquely determined by the element elastic nodal displacements, i.e.,

$$\underline{S} = \underline{k}^E \ \underline{r}^E \tag{21}$$

and therefore

$$d\underline{S} = \underline{k}^E \ d\underline{r}^E \tag{22}$$

where $\underline{k}^E$ is the elastic stiffness matrix of the element.

According to the associated flow rule for an elasto-plastic solid [24], the plastic deformation increments $d\underline{r}_K^P$ at a particular section K, is governed by

$$d\underline{r}_K^P = \left[\frac{\partial f}{\partial \underline{S}_K}\right] d\lambda_K \tag{23}$$

where $d\lambda_K$ is a positive scalar proportionality constant which can be determined by the yield condition, i.e.,

$$f(\underline{S}_K) = 1 \quad , \qquad df = \left[\frac{\partial f}{\partial \underline{S}_K}\right]^T d\underline{S}_K = 0 \tag{24}$$

where T indicates the transpose operation. Note that if the section is not yielding, Eq. (23) still applies provided $\partial f/\partial \underline{S}_K$ is set equal to $\underline{0}$. Letting K = I, J, the flow rule can be expressed as

$$d\underline{r}^P = \begin{Bmatrix} d\underline{r}_I^P \\ \\ d\underline{r}_J^P \end{Bmatrix} = \begin{bmatrix} \dfrac{\partial f}{\partial \underline{S}_I} & \underline{0} \\ \\ \underline{0} & \dfrac{\partial f}{\partial \underline{S}_J} \end{bmatrix} \begin{Bmatrix} d\lambda_I \\ \\ d\lambda_J \end{Bmatrix} \equiv \left[\frac{\partial f}{\partial \underline{S}}\right] d\underline{\lambda} \tag{25}$$

and the yield condition becomes

$$\underset{\sim}{f} = \begin{Bmatrix} f(\underset{\sim}{S}_I) \\ \\ f(\underset{\sim}{S}_J) \end{Bmatrix} = \begin{Bmatrix} 1 \\ \\ 1 \end{Bmatrix}, \quad d\underset{\sim}{f} = \begin{bmatrix} \frac{\partial f}{\partial \underset{\sim}{S}_I} & \underset{\sim}{o} \\ \underset{\sim}{o} & \frac{\partial f}{\partial \underset{\sim}{S}_J} \end{bmatrix}^T \begin{Bmatrix} d\underset{\sim}{S}_I \\ \\ d\underset{\sim}{S}_J \end{Bmatrix}$$

$$= \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T d\underset{\sim}{S} = \underset{\sim}{o} \tag{26}$$

Premultiplying both sides of the equation

$$d\underset{\sim}{S} = \underset{\sim}{k}^E \, d\underset{\sim}{r}^E = \underset{\sim}{k}^E \, (d\underset{\sim}{r} - d\underset{\sim}{r}^P) \tag{27}$$

by $[\partial f/\partial \underset{\sim}{S}]^T$, one obtains upon using Eq. (26) the relation

$$\left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T d\underset{\sim}{S} = \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \, (d\underset{\sim}{r} - d\underset{\sim}{r}^P) = \underset{\sim}{o} \tag{28}$$

Substituting Eq. (25) into Eq. (28), one obtains

$$\left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right] d\underset{\sim}{\lambda} = \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \, d\underset{\sim}{r} \tag{29}$$

Solving for $d\underset{\sim}{\lambda}$ gives

$$d\underset{\sim}{\lambda} = \left\{ \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right] \right\}^{-1} \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \quad d\underset{\sim}{r} \tag{30}$$

which after substituting into Eq. (25) gives

$$d\underset{\sim}{r}^P = \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right] \left\{ \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right] \right\}^{-1} \left[\frac{\partial f}{\partial \underset{\sim}{S}} \right]^T \underset{\sim}{k}^E \quad d\underset{\sim}{r} \tag{31}$$

Substituting Eq. (31) into Eq. (27) the required relation

$$d\underset{\sim}{S} = (\underset{\sim}{k}^E - \underset{\sim}{k}_t^Y) \, d\underset{\sim}{r} \equiv \underset{\sim}{k}_t^{EP} \, d\underset{\sim}{r} \tag{32}$$

is obtained where

$$\underline{k}_t^Y \equiv \underline{k}^E \left[\frac{\partial f}{\partial \underline{S}}\right] \left\{ \left[\frac{\partial f}{\partial \underline{S}}\right]^T \underline{k}^E \left[\frac{\partial f}{\partial \underline{S}}\right] \right\}^{-1} \left[\frac{\partial f}{\partial \underline{S}}\right]^T \underline{k}^E \tag{33}$$

Physically speaking, the matrix $\underline{k}_t^Y$ represents the losses of elastic stiffness in the element due to yielding, and the elasto-plastic tangent stiffness matrix $\underline{k}_t^{EP}$ represents the reduced elastic stiffnesses of the element. It should be mentioned that matrix $\underline{k}_t^{EP}$ may be singular depending upon the location of the point of interest on the yield surface.

E. COMPUTATIONAL TECHNIQUES

The evaluation of the elasto-plastic tangent stiffness matrix appearing in Eq. (33) is straight forward theoretically once the yield point representing the element force state on yield surface is determined. However, in practice a step-by-step analysis procedure is usually required in which case the yield point determination is only approximate. To be more specific, a point inside the yield surface can move outside the surface during a single time interval provided the element is in a yielding state.

To treat this problem correctly, it is necessary to use an iteration procedure within each time interval. This procedure however requires a tremendous amount of computational effort. Therefore, in the present analysis, an approximate procedure which avoids iteration is adopted. This procedure is similar to one used by Chi and Powell [25].

Referring to Fig. 11, assume that the column element at the beginning of a time interval Δt and at time t is in the elastic state as represented by point A inside the yield surface. Following the established procedures a vector of element nodal displacement increments $\Delta\underline{r}_t$ is determined and an apparent new element force vector $\bar{\underline{S}}_{t+\Delta t}$ is then found using the relations

$$\bar{\underline{S}}_{t+\Delta t} = \underline{S}_t + \Delta\underline{S}_t^E = \underline{S}_t + \underline{k}^E \Delta\underline{r}_t \tag{34}$$

and

$$f(\bar{\underline{S}}_{t+\Delta t}) > 1 \tag{35}$$

The method then used to determine yield point C proceeds by determining a scalar factor μ_1, $o \leq \mu_1 \leq 1$ such that $f(\underline{S}_t + \mu_1 \Delta \underline{S}_t^E) = 1$. One can then evaluate the elasto-plastic tangent stiffness matrix for point C. A new element force vector $\bar{\bar{\underline{S}}}_{t+\Delta t}$, as represented by point D, is then computed using the relation

$$\bar{\bar{\underline{S}}}_{t+\Delta t} = \underline{S}_t + \mu_1 \Delta \underline{S}_t^E + (1-\mu_1) \underline{k}_t^{EP} \Delta \underline{r}_t \tag{36}$$

Usually, point D is also located outside the yield surface due to the local convex shape of the yield surface. A second scalar factor μ_2 is now introduced to bring point D to point E on the yield surface along the vector $\bar{\bar{\underline{S}}}_{t+\Delta t}$. The final element force vector is determined by

$$\underline{S}_{t+\Delta t} = \mu_2 \bar{\bar{\underline{S}}}_{t+\Delta t} \quad , \quad 0 \leq \mu_2 \leq 1 \tag{37}$$

where μ_2 is determined so that $f(\mu_2 \bar{\bar{\underline{S}}}_{t+\Delta t}) = 1$.

Finally, at the end of the time interval, a new elasto-plastic tangent stiffness matrix is evaluated, using the new yielding force vector $\underline{S}_{t+\Delta t}$, i.e., the vector to point E. This new tangent stiffness matrix is then used as the element stiffness matrix during the next time interval.

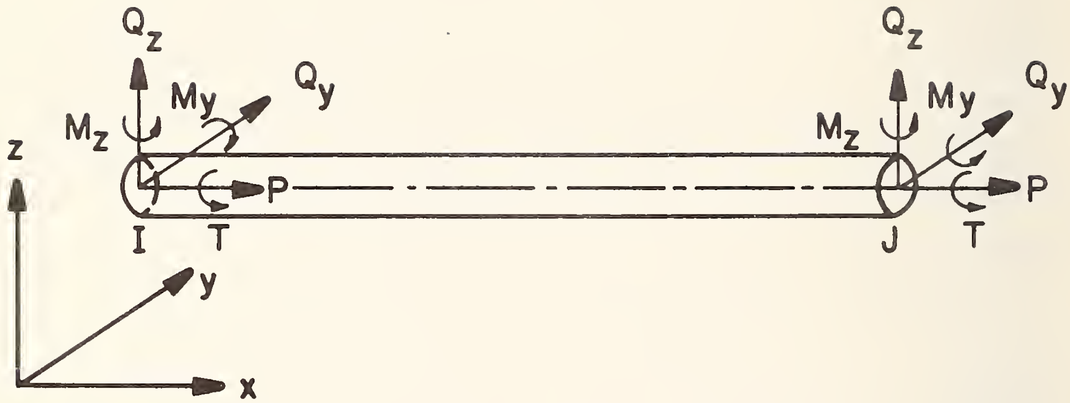


Fig. 5 Local Coordinate System For a Bridge Column

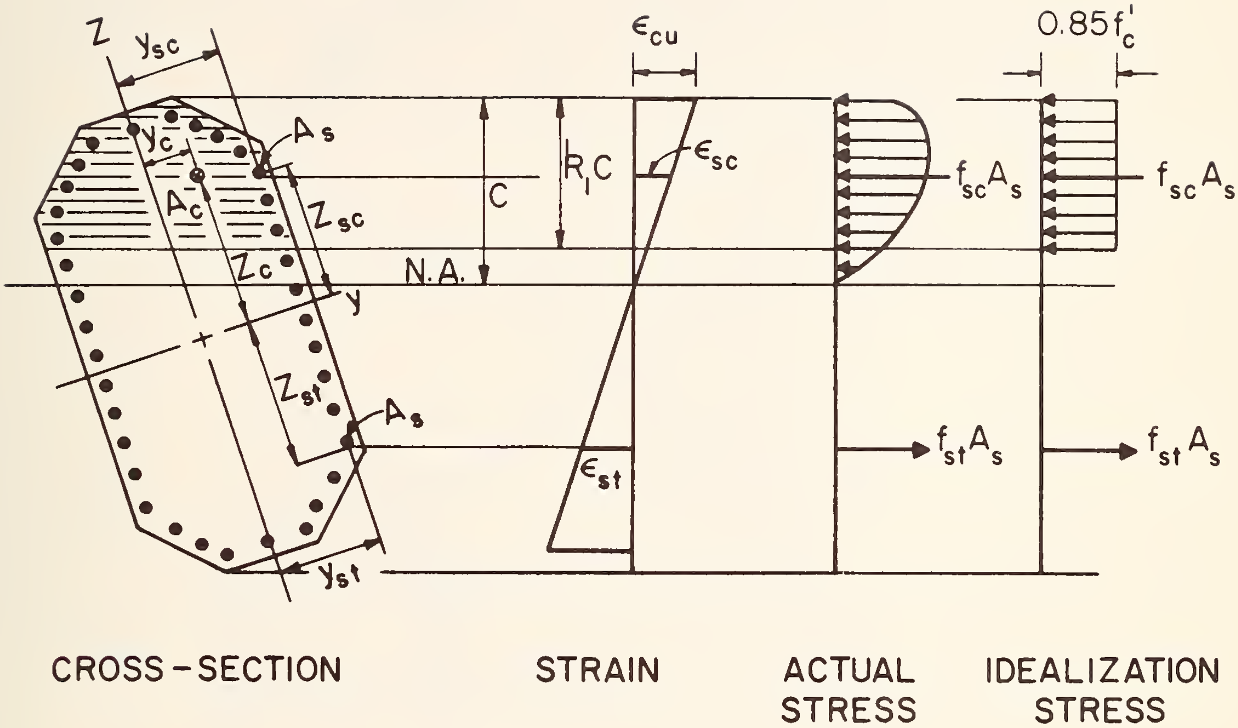
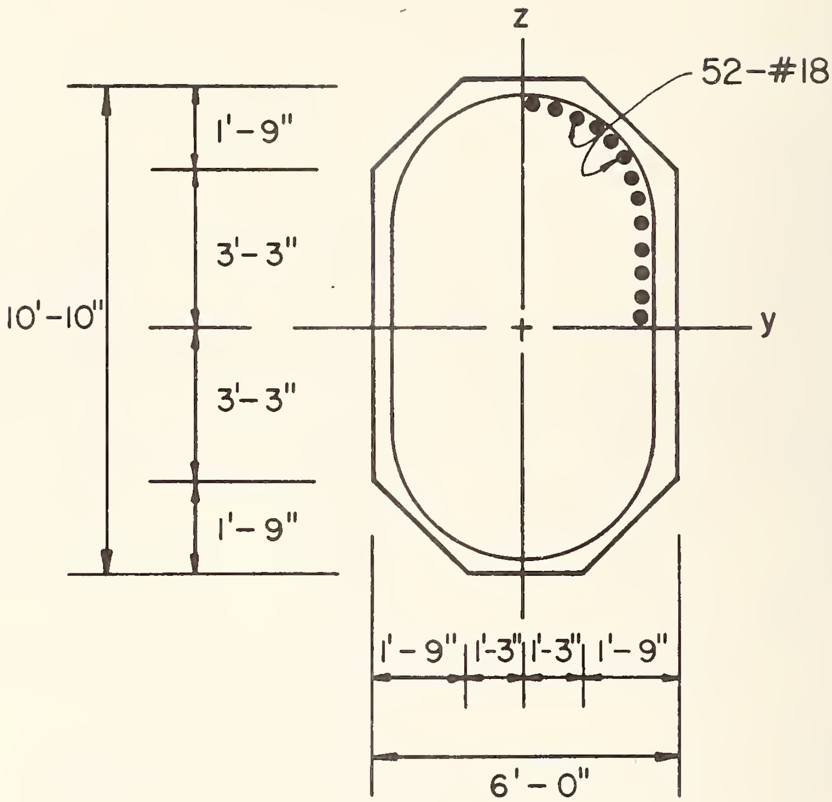


Fig. 6 Idealized Ultimate Stress-Strain Distribution For a Typical Bridge Column Section



$f'_c = 4{,}000$ PSI

$f'_{sy} = 40{,}000$ PSI

$E_s = 2.9 \times 10^7$ PSI

$P_o = 3.47 \times 10^7$ LB.

$M_{yo} = 3.99 \times 10^8$ LB.-IN

$M_{zo} = 2.58 \times 10^8$ LB.-IN

Fig. 7 Cross-Section of a Typical Bridge Column

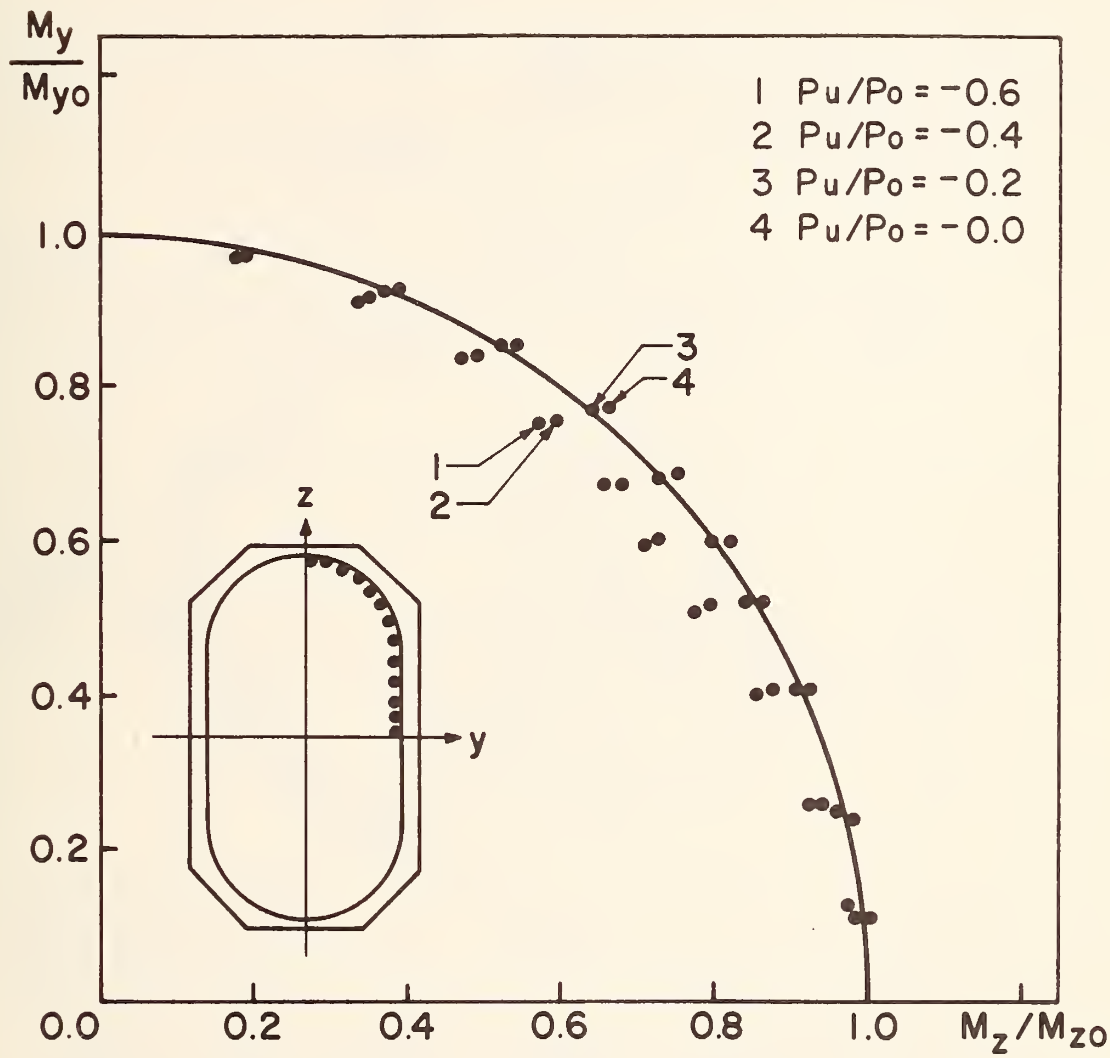


Fig. 8 Biaxial Bending Interaction Curve

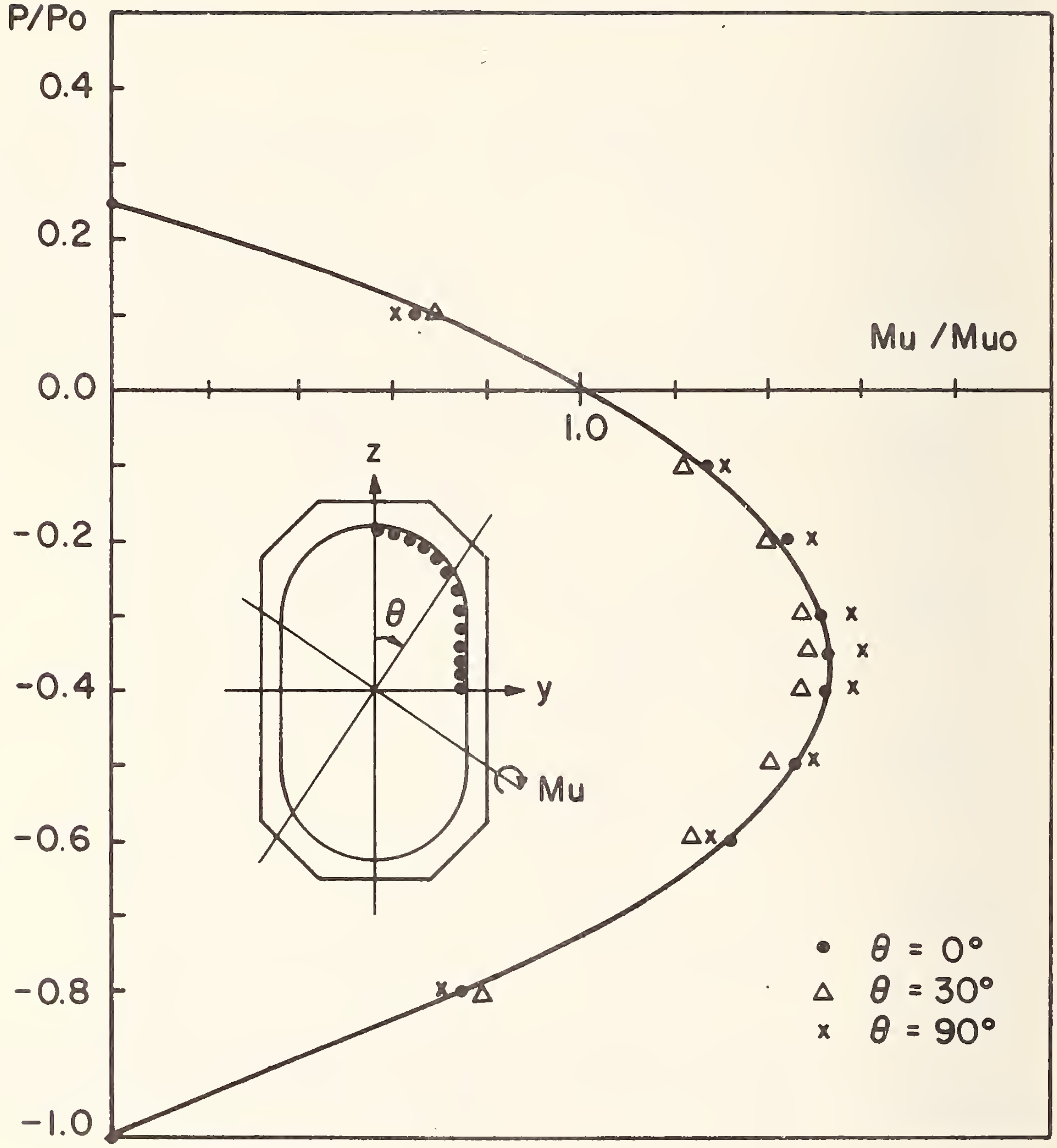


Fig. 9 Axial Force and Bending Moment Interaction Curve

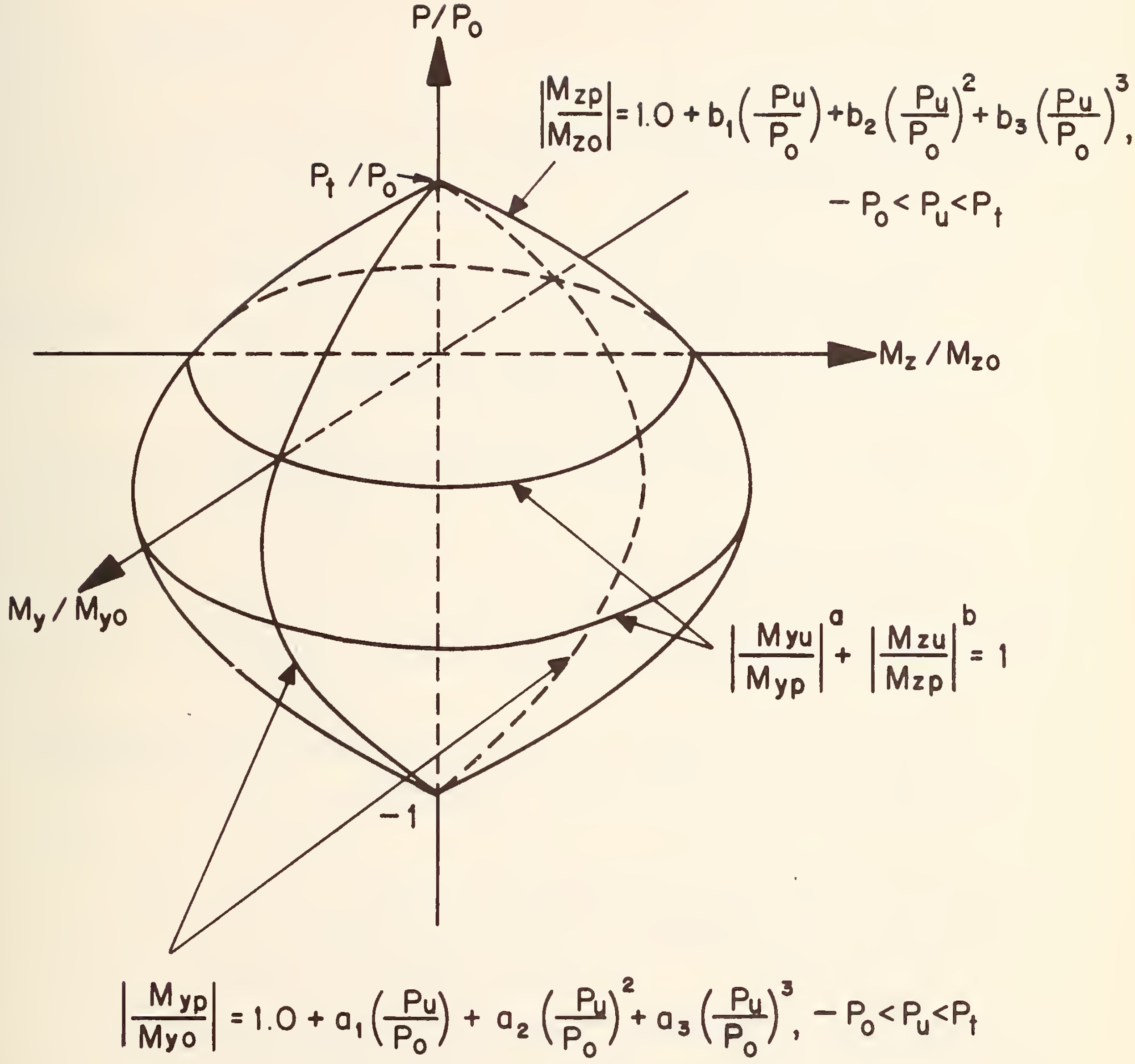


Fig. 10 Yield Surface of Typical Bridge Column

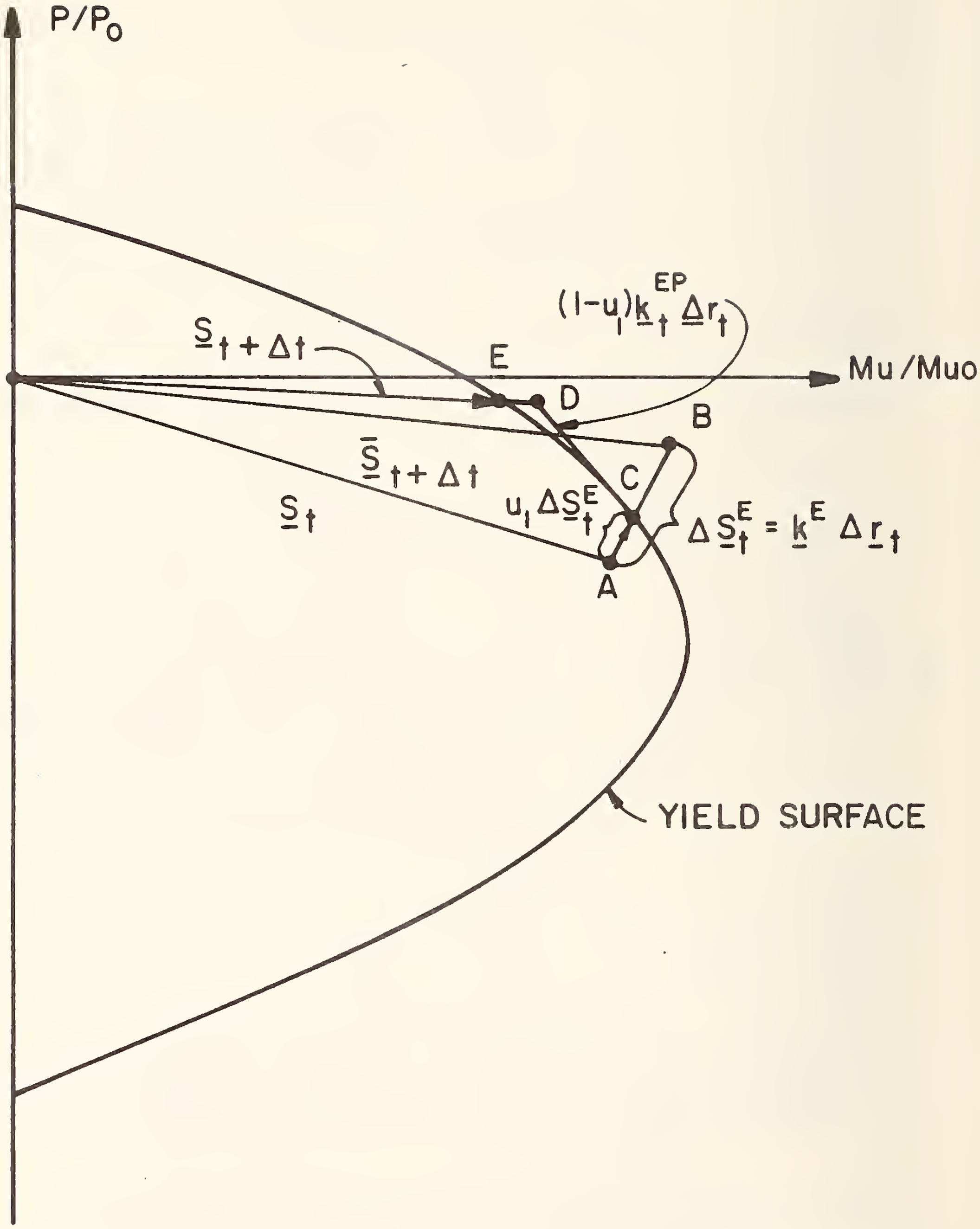


Fig. 11 Determination of Yield Points

IV NONLINEAR MODEL FOR EXPANSION JOINTS

A. IDEALIZATION

A nonlinear mathematical model for simulating the dynamic behavior of expansion joints has been defined. This model is idealized as follows (see Fig. 12):

(1) The end diaphragms of the deck at each expansion joint are rigid.

(2) Contact within the expansion joint can develop at only two points, A and B, located at the ends of the rigid diaphragms and separated by a distance d in the transverse direction (y axis).

(3) At contact points A and B, longitudinal (x axis) impact springs having a large stiffness* k_I are attached to one rigid diaphragm leaving a small gap Δ_G between the other diaphragm and the springs.

(4) Longitudinal impact starts at points A and B when the relative displacements of the two rigid diaphragms close the gap Δ_G.

(5) Relative slippage between the two diaphragms can take place in the longitudinal direction at points A and B under the action of Coulomb friction.

(6) During periods of slippage, the Coulomb friction forces at points A and B are proportional to their respective vertical contact forces and act in directions opposite to their directions of slippage.

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* The larger the stiffness is, the closer that true impact condition is attended. The practical limit of this value depends upon the number of significant digit carried by the computer used and the time interval chosen in the analysis.

(7) A total of N_T longitudinal restrainer ties having equal elastic tensile stiffnesses k_T and zero compressive stiffnesses are placed through the two rigid diaphragms with a tie gap Δ_T at one end of each bar and at transverse distance $y_i\,(i=1,2,---,N_T)$ from the longitudinal center line axis of the deck.

(8) Longitudinal restrainer ties are elasto-plastic in tension and have equal yield forces, S_T.

(9) Relative transverse motions of the two rigid diaphragms are restrained by a very stiff elastic shear spring of stiffness k_S located at the center point C.

(10) At contact points A and B, the rigid diaphragms are interconnected by vertical (z axis) springs having stiffnesses equal to k_V.

(11) To provide generality in the model, the expansion joint may be skewed with respect to the deck, i.e., the transverse axis of the point (s axis; Fig. 13) may not be normal to the x axis.

The behavior of the idealized expansion joint shown in Fig. 12 can be characterized conveniently using displacement coordinates, $\bar{\underset{\sim}{r}}$, as shown in Fig. 13, i.e.

$$\bar{\underset{\sim}{r}} = \begin{Bmatrix} \bar{\underset{\sim}{r}}_I \\ \\ \bar{\underset{\sim}{r}}_J \end{Bmatrix} \quad ; \quad \bar{\underset{\sim}{r}}_K = \begin{bmatrix} r_{Ax} \\ r_y \\ r_{Az} \\ r_{Bx} \\ \theta_s \\ r_{Bz} \end{bmatrix}_K \qquad (K = I,J) \qquad (38)$$

where θ_s is a joint rotation about the s axis which may be skewed by an angle ψ as shown. The expansion joint coordinates $\bar{\underset{\sim}{r}}$ can be related to the local nodal coordinates $\underset{\sim}{r}$ as defined by

$$\underline{r} = \begin{Bmatrix} \underline{r}_I \\ \\ \underline{r}_J \end{Bmatrix} \qquad \underline{r}_K = \begin{bmatrix} r_x \\ r_y \\ r_z \\ \theta_x \\ \theta_y \\ \theta_z \end{bmatrix}_K \qquad (K = I,J) \tag{39}$$

through the following transformation:

$$\bar{\underline{r}} = \begin{Bmatrix} \bar{\underline{r}}_I \\ \\ \bar{\underline{r}}_J \end{Bmatrix} = \begin{bmatrix} \underline{a} & \underline{0} \\ \\ \underline{0} & \underline{a} \end{bmatrix} \begin{Bmatrix} \underline{r}_I \\ \\ \underline{r}_J \end{Bmatrix} \equiv \underline{A}\ \underline{r} \tag{40}$$

where transformation matrix $\underline{a}$ is given by

$$\underline{a} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & \frac{d}{2} \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & -\frac{d}{2} & \frac{d}{2}\tan\psi & 0 \\ 1 & 0 & 0 & 0 & 0 & -\frac{d}{2} \\ 0 & 0 & 0 & 0 & \frac{1}{\cos\psi} & 0 \\ 0 & 0 & 1 & \frac{d}{2} & -\frac{d}{2}\tan\psi & 0 \end{bmatrix} . \tag{41}$$

The forces associated with the expansion joint coordinates are

$$\bar{\underline{S}} = \begin{Bmatrix} \bar{\underline{S}}_I \\ \\ \bar{\underline{S}}_J \end{Bmatrix} \qquad \bar{\underline{S}}_K = \begin{bmatrix} S_{Ax} \\ S_y \\ S_{Az} \\ S_{Bx} \\ M_s \\ S_{Bz} \end{bmatrix}_K \quad , \ (K = I,J) \tag{42}$$

while the forces associated with the local nodal coordinates are:

$$\underline{S} = \begin{Bmatrix} \underline{S}_I \\ \\ \underline{S}_J \end{Bmatrix} \qquad \underline{S}_K = \begin{bmatrix} S_x \\ S_y \\ S_z \\ M_x \\ M_y \\ M_z \end{bmatrix}_K \qquad (K = I,J) \tag{43}$$

It is easy to show that $\bar{\underline{S}}$ are related to $\underline{S}$ by the transformation

$$\underline{S} = \begin{Bmatrix} \underline{S}_I \\ \\ \underline{S}_J \end{Bmatrix} = \begin{bmatrix} \underline{a}^T & \underline{0} \\ \\ \underline{0} & \underline{a}^T \end{bmatrix} \begin{Bmatrix} \bar{\underline{S}}_I \\ \\ \bar{\underline{S}}_J \end{Bmatrix} \equiv \underline{A}^T \bar{\underline{S}} \tag{44}$$

B. PIECEWISE LINEAR EXPANSION JOINT STIFFNESSES

The idealized expansion joint can be characterized by a stiffness matrix which is piecewise linear with a number of discontinuities depending upon the relative displacements of the joint. For a step-by-step dynamic analysis, if it is assumed that these finite number of discontinuities occur at the end of each time step, then the vector of expansion joint force increments $\Delta\bar{\underline{S}}$ can be related to the vector of the expansion joint displacement increments $\Delta\bar{\underline{r}}$ by a stiffness matrix $\bar{\underline{k}}_t^{EJ}$ which is constant within each time step, i.e.,

$$\Delta\bar{\underline{S}} = \bar{\underline{k}}_t^{EJ} \, \Delta\bar{\underline{r}} \tag{45}$$

where $\bar{\underline{k}}_t^{EJ}$ is a function of $\bar{\underline{r}}$ at time t.

It is convenient to define the stiffness coefficients of $\bar{\underline{k}}_t^{EJ}$ in terms of the relative expansion joint displacements $\bar{\underline{u}}$ defined by

$$\underline{\bar{u}} = \begin{bmatrix} u_{Ax} \\ u_y \\ u_{Az} \\ u_{Bx} \\ u_S \\ u_{Bz} \end{bmatrix} \equiv \begin{bmatrix} r_{Ax} \\ r_y \\ r_{Az} \\ r_{Bx} \\ \theta_S \\ r_{Bz} \end{bmatrix}_J - \begin{bmatrix} r_{Ax} \\ r_y \\ r_{Az} \\ r_{Bx} \\ \theta_S \\ r_{Bz} \end{bmatrix}_I = \underline{\bar{r}}_J - \underline{\bar{r}}_I \tag{46}$$

and the corresponding expansion joint forces $\underline{\bar{F}}$ defined by

$$\underline{\bar{F}} = \begin{bmatrix} F_{Ax} \\ F_y \\ F_{Az} \\ F_{Bx} \\ F_S \\ F_{Bz} \end{bmatrix} = \begin{bmatrix} S_{Ax} \\ S_y \\ S_{Az} \\ S_{Bx} \\ M_S \\ S_{Bz} \end{bmatrix}_J = \underline{\bar{S}}_J = -\underline{\bar{S}}_I \tag{47}$$

Thus, the idealized expansion joint can be characterized by a stiffness matrix $\underline{\bar{k}}$ relating $\Delta\underline{\bar{F}}$ to $\Delta\underline{\bar{u}}$, i.e.,

$$\Delta\underline{\bar{F}} = \underline{\bar{k}}\ \Delta\underline{\bar{u}} \tag{48}$$

Stiffness matrix $\underline{\bar{k}}$ represents contributions from two sources. One contribution is the stiffness matrix $\underline{\bar{k}}_1$ of the idealization expansion joint without longitudinal restrainer ties and the other contribution is the matrix $\underline{\bar{k}}_2$ due to the presence of longitudinal ties, i.e.,

$$\underline{\bar{k}} = \underline{\bar{k}}_1 + \underline{\bar{k}}_2 \tag{49}$$

The stiffness coefficients of $\underline{\bar{k}}_1$ can be written in the form

$$\bar{\underset{\sim}{k}}_1 = \begin{bmatrix} k_A & 0 & 0 & 0 & 0 & 0 \\ 0 & k_S & 0 & 0 & 0 & 0 \\ 0 & 0 & k_V & 0 & 0 & 0 \\ 0 & 0 & 0 & k_B & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & k_V \end{bmatrix} \tag{50}$$

where

$$k_A \begin{cases} = k_I & \text{for} \quad u_{Ax} < -\Delta_G \ \text{and} \ \dot{u}_{Ax} \leq 0, \\ = 0 & \text{for} \quad u_{Ax} < -\Delta_G \ \text{and} \ \dot{u}_{Ax} > 0, \\ & \text{or} \quad u_{Ax} \geq -\Delta_G \end{cases}$$

$$k_B \begin{cases} = k_I & \text{for} \quad u_{Bx} < -\Delta_G \ \text{and} \ \dot{u}_{Bx} \leq 0, \\ = 0 & \text{for} \quad u_{Bx} < -\Delta_G \ \text{and} \ \dot{u}_{Bx} > 0, \\ & \text{or} \quad u_{Bx} \geq -\Delta_G \end{cases} \tag{51}$$

and where k_S, k_V are those quantities defined previously.

The stiffness coefficient of each longitudinal restrainer tie within a time step can be expressed as

$$\Delta F_{Ti} = k_{Ti} \Delta u_{Ti} \quad , \quad (i=1,---,N_T) \tag{52}$$

where ΔF_{Ti} and Δu_{Ti} are, respectively, the increment of tie force F_{Ti} and the increment of tie deformation u_{Ti} in the ith tie during each time interval. The stiffness k_{Ti} is the instantaneous stiffness of the ith tie bar at time t, which can be expressed as

$$
\begin{aligned}
& = 0 \quad \text{for} \quad u_{Ti} \le (\Delta_T + u^P_{Ti}) \quad , \\
k_{Ti} & = k_T \quad \text{for} \quad (\Delta_T + u^P_{Ti}) < u_{Ti} \le (\Delta_T + u^P_{Ti} + u^E_T), \\
& = 0 \quad \text{for} \quad u_{Ti} > (\Delta_T + u^P_{Ti} + u^E_T).
\end{aligned}
\tag{53}
$$

where $u^P_{Ti} \geq 0$ is the current plastic elongation of the ith tie, and u^E_T is the elastic elongation of the ties, i.e., $u^E_T = S_T/k_T$.

Since the end diaphragms are rigid, the tie deformation u_{Ti} can be determined from the longitudinal expansion joint deformations u_{Ax} and u_{Bx} by the relation

$$
u_{Ti} = \left\langle \frac{1}{2} + \frac{2y_i}{d} \quad , \quad \frac{1}{2} - \frac{2y_i}{d} \right\rangle \begin{Bmatrix} u_{Ax} \\ u_{Bx} \end{Bmatrix} , \quad (i=1,2,---,N_T) \tag{54}
$$

From conditions of equilibrium, the longitudinal expansion joint forces F_{Ax} and F_{Bx} can be expressed as the sum of all tie bar forces F_{Ti}, i.e.,

$$
\begin{Bmatrix} F_{Ax} \\ F_{Bx} \end{Bmatrix} = \sum_{i=1}^{N_T} \begin{Bmatrix} \frac{1}{2} + \frac{2y_i}{d} \\ \frac{1}{2} - \frac{2y_i}{d} \end{Bmatrix} F_{Ti} \tag{55}
$$

Combining Eq. (52), (53), and (54), the following relation is obtained

$$
\begin{Bmatrix} F_{Ax} \\ F_{Bx} \end{Bmatrix} = \begin{bmatrix} k_{AA} & k_{AB} \\ k_{AB} & k_{BB} \end{bmatrix} \begin{Bmatrix} u_{Ax} \\ u_{Bx} \end{Bmatrix} \tag{56}
$$

where

$$
\begin{aligned}
k_{AA} &= \sum_{i=1}^{N_T} k_{Ti} \left(\frac{1}{2} + \frac{2y_i}{d} \right)^2 \\
k_{AB} &= \sum_{i=1}^{N_T} k_{Ti} \left(\frac{1}{2} + \frac{2y_i}{d} \right) \left(\frac{1}{2} - \frac{2y_i}{d} \right) \\
k_{BB} &= \sum_{i=1}^{N_T} k_{Ti} \left(\frac{1}{2} + \frac{2y_i}{d} \right)^2
\end{aligned}
\tag{57}
$$

Thus, the stiffness matrix $\bar{\underline{k}}_2$ in Eq. (44) can be expressed as

$$
\bar{\underline{k}}_2 = \begin{bmatrix}
k_{AA} & 0 & 0 & k_{AB} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 \\
k_{AB} & 0 & 0 & k_{BB} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0
\end{bmatrix}
\tag{58}
$$

Using Eqs. (46) and (47), the stiffness matrix $\bar{\underline{k}}_t^{EJ}$ can now be expressed as

$$
\bar{\underline{k}}_t^{EJ} = \begin{bmatrix}
\bar{\underline{k}} & -\bar{\underline{k}} \\
-\bar{\underline{k}} & \bar{\underline{k}}
\end{bmatrix}
\tag{59}
$$

Finally, the stiffness matrix $\bar{\underline{k}}_t^{EJ}$ must be transformed to the local nodal coordinate system using Eqs. (40) and (44). This transformation results in the relation

$$
\Delta \underline{S} = \underline{k}_t^{EJ} \, \Delta \underline{r} \quad , \quad \underline{k}_t^{EJ} = \underline{A}^T \, \bar{\underline{k}}_t^{EJ} \, \underline{A}
\tag{60}
$$

Matrix $\underline{k}_t^{EJ}$ is the stiffness matrix which relates the vector of local nodal force increments $\Delta \underline{S}$ to the vector of local nodal displacement increments $\Delta \underline{r}$ at time t, and which, after local to global coordinate transformation, can be used to assemble the total stiffness $\underline{K}_t$ for the complete structural system.

At the end of each time interval during the analysis process, joint conditions as expressed by Eq. (51) and the condition of each longitudinal restrainer tie are checked and expansion joint stiffnesses are formed accordingly. The quantities, u_{Ax}, u_{Bx}, and u_{Ti}^P, $(i=1,2,\text{---},N_T)$ are carefully monitored since the maximum value of u_{Ax} and u_{Bx} represent the maximum expansion joint separations at points A and B and the maximum values of u_{Ti}^P, $(i=1,2,\text{---},N_T)$ indicate the maximum ductilities that the longitudinal restrainer ties should provide.

C. COULOMB FRICTION FORCES

Coulomb friction forces C_{Ax} and C_{Bx}, which develop at contact points A and B when the expansion joint undergoes longitudinal relative displacements and when the vertical contact forces F_{Az} and F_{Bz} are compressive, can be considered as pairs of self-equilibrating forces acting on the expansion joint along the longitudinal x axis at points A and B. According to the assumptions stated previously, these forces can be expressed as

$$\begin{aligned} C_{Ax} &= \nu \langle F_{Az} \rangle \left| F_{Az} \right| \operatorname{sign} (\dot{u}_{Ax}) \\ C_{Bx} &= \nu \langle F_{Bz} \rangle \left| F_{Bz} \right| \operatorname{sign} (\dot{u}_{Bx}) \end{aligned} \qquad (61)$$

where ν is a constant coefficient of friction and where

$$\langle F_{Az} \rangle \begin{array}{ll} = 1 & F_{Az} < 0 \\ = 0 & F_{Az} \geq 0 \end{array} \qquad (62)$$

$$\langle F_{Bz} \rangle \begin{array}{ll} = 1 & F_{Bz} < 0 \\ = 0 & F_{Bz} \geq 0 \end{array} \qquad (63)$$

If it is assumed that the relative velocities $\dot{u}_{Ax}$ and $\dot{u}_{Bx}$, and the forces F_{Az} and F_{Bz} do not change signs during a time interval Δt, then the changes of Coulomb friction forces during the interval can be expressed by

$$\begin{aligned} \Delta C_{Ax} &= \nu \langle F_{Az} \rangle \,|\, \Delta F_{Az} \,|\, \text{sign}\,(\dot{u}_{Ax}) \\ \Delta C_{Bx} &= \nu \langle F_{Bz} \rangle \,|\, \Delta F_{Bz} \,|\, \text{sign}\,(\dot{u}_{Bx}) \end{aligned} \tag{64}$$

Let $\Delta \underline{\bar{P}}^C$ represent the incremental Coulomb friction force vector in the expansion joint coordinate system during time interval Δt. Then $\Delta \underline{\bar{P}}^C$ can be expressed as

$$\Delta \underline{\bar{P}}^C = \begin{Bmatrix} \Delta \underline{\bar{P}}_I^C \\ \Delta \underline{\bar{P}}_J^C \end{Bmatrix}, \quad \Delta \underline{\bar{P}}_I^C = \begin{bmatrix} \Delta C_{Ax} \\ 0 \\ 0 \\ \Delta C_{Bx} \\ 0 \\ 0 \end{bmatrix}, \quad \Delta \underline{\bar{P}}_J^C = \begin{bmatrix} -\Delta C_{Ax} \\ 0 \\ 0 \\ -\Delta C_{Bx} \\ 0 \\ 0 \end{bmatrix} \tag{65}$$

Thus, the incremental Coulomb friction force vector $\Delta \underline{P}^C$ in the local nodal coordinate system can be obtained using Eq. (44); i.e.

$$\Delta \underline{P}^C = \underline{A}^T \, \Delta \underline{\bar{P}}^C \tag{66}$$

When $\Delta \underline{P}^C$ for all expansion joints in the bridge system are transformed to the global coordinate system and are assembled, an incremented Coulomb friction force vector $\Delta \underline{R}^C$ in the global coordinate system is obtained. The vectors $\Delta \underline{R}^C$ are then added to the incremental external force vectors during each time interval.

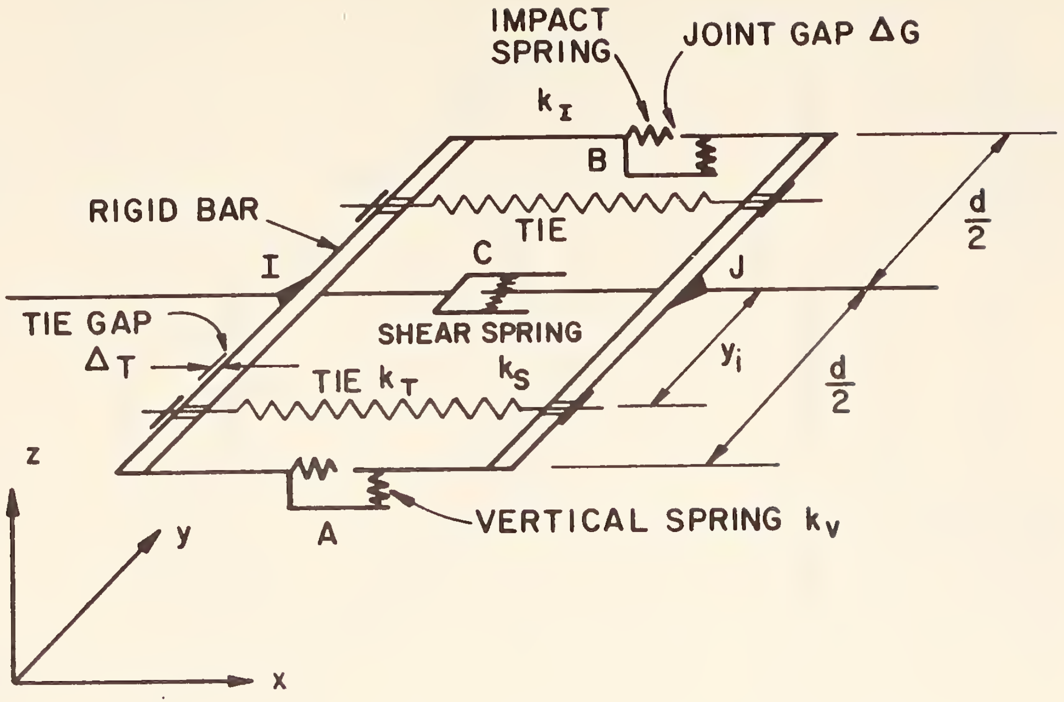


Fig. 12 Idealized Bridge Expansion Joint

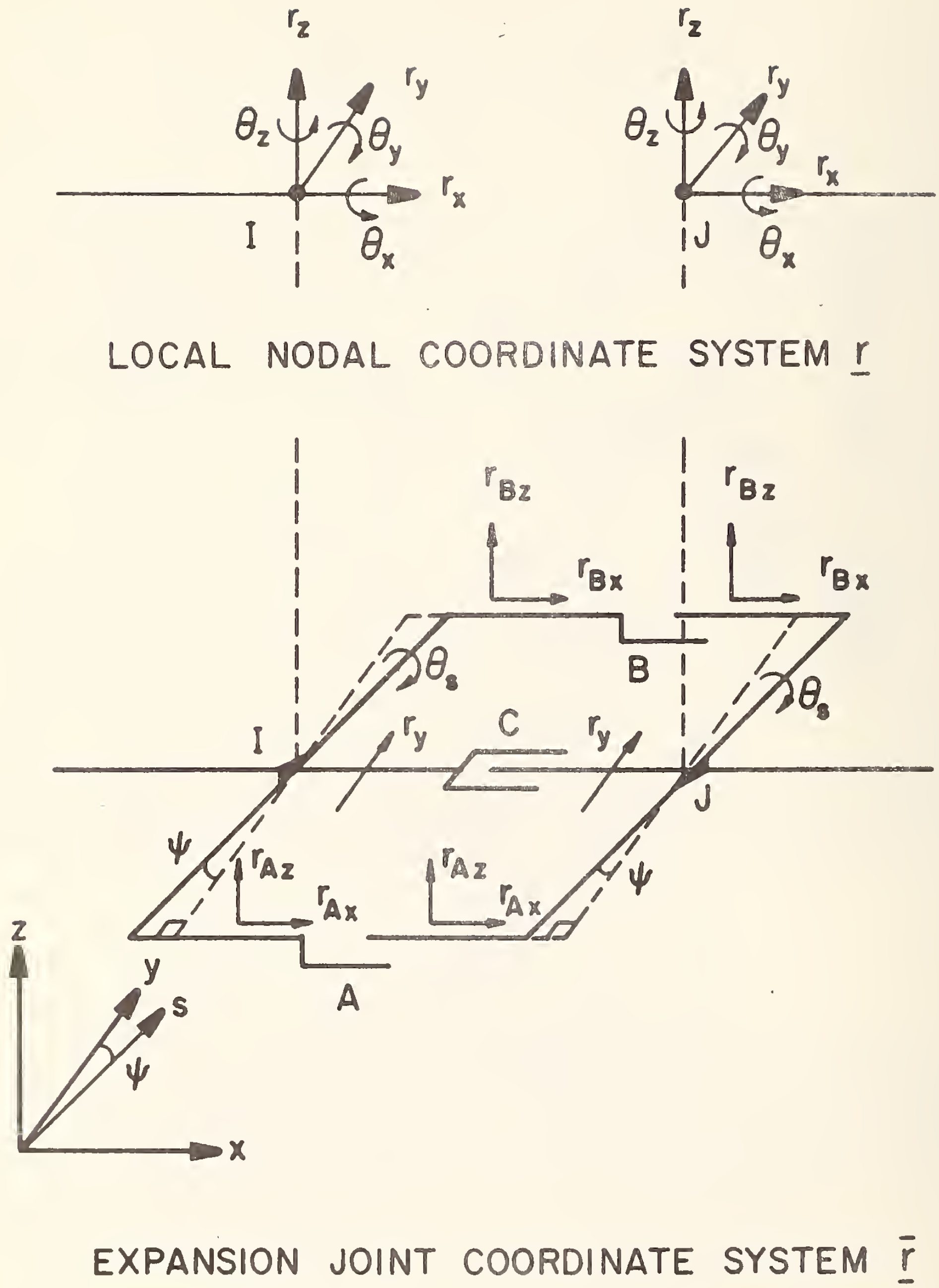


Fig. 13 Coordinate Systems For Expansion Joint

A. EQUATIONS OF MOTION

The equations of motion for an n degree of freedom system expressing dynamic equilibrium at time t can be expressed in the standard matrix form

$$\underline{M}\,\ddot{\underline{u}}_t + \underline{C}_t\dot{\underline{u}}_t + \underline{K}_t\underline{u}_t = \underline{R}(t) \tag{67}$$

where $\underline{M}$, $\underline{C}_t$, and $\underline{K}_t$ are the mass, damping, and stiffness matrices, respectively, and where $\underline{R}(t)$ is the applied dynamic load vectors. Vectors $\ddot{\underline{u}}_t$, $\dot{\underline{u}}_t$, and $\underline{u}_t$ are the absolute acceleration, velocity, and displacement vectors, respectively, as measured with respect to a fixed set of global coordinate axes.

If the system is subjected to prescribed support excitations, a complete set of nodal displacements $\underline{u}_t^c$ should be considered which include, in addition to the n free nodal displacements, the n^b prescribed non-zero support displacements. Thus, the complete nodal displacement vector can be expressed in the partitioned form

$$\underline{u}_t^c = \begin{Bmatrix} \underline{u}_t \\ \underline{u}_t^b \end{Bmatrix} \tag{68}$$

where $\underline{u}_t^b$ is a vector containing the n^b non-zero support displacements (translations and/or rotations). Vector $\underline{u}_t^c$ can be conveniently decomposed into a quasi-static displacement vector $\underline{u}_s^c$ and a dynamic displacement vector $\underline{u}^c$, i.e.

$$\underline{u}_t^c = \begin{Bmatrix} \underline{u}_t \\ \\ \underline{u}_t^b \end{Bmatrix} = \begin{Bmatrix} \underline{u}_s \\ \\ \underline{u}_s^b \end{Bmatrix} + \begin{Bmatrix} \underline{u} \\ \\ \underline{u}^b \end{Bmatrix} \equiv \underline{u}_s^c + \underline{u}^c \tag{69}$$

where by definition $\underline{u}^b = \underline{0}$. Enlarging the mass, damping, and stiffness matrices as well as the dynamic load vector in Eq. (67) to account for the n^b support displacements, the equations of motion for the complete

system can be expressed in the partitioned matrix form

$$\left[\begin{array}{c|c} \underline{M} & \underline{M}^b \\ \hline (\underline{M}^b)^T & \underline{M}^{bb} \end{array}\right] \left\{\begin{array}{c} \ddot{\underline{u}}_t \\ \hline \ddot{\underline{u}}_t^b \end{array}\right\} + \left[\begin{array}{c|c} \underline{C}_t & \underline{C}_t^b \\ \hline (\underline{C}_t^G)^T & \underline{C}_t^{bb} \end{array}\right] \left\{\begin{array}{c} \dot{\underline{u}}_t \\ \hline \dot{u}_t^b \end{array}\right\} + \left[\begin{array}{c|c} \underline{K}_t & \underline{K}_t^b \\ \hline (K_t^b)^T & \underline{K}_t^{bb} \end{array}\right] \left\{\begin{array}{c} \underline{u}_t \\ \hline \underline{u}_t^b \end{array}\right\}$$

$$= \left\{\begin{array}{c} \underline{R}(t) \\ \hline \underline{R}^b(t) \end{array}\right\} \tag{70}$$

The equations of motion associated with the n free nodal displacements now become

$$\left[\underline{M}\ \underline{M}^b\right] \left\{\begin{array}{c} \ddot{\underline{u}}_t \\ \hline \ddot{\underline{u}}_t^b \end{array}\right\} + \left[\underline{C}_t\ \underline{C}_t^b\right] \left\{\begin{array}{c} \dot{\underline{u}}_t \\ \hline \dot{\underline{u}}_t^b \end{array}\right\} + \left[\underline{K}_t\ \underline{K}_t^b\right] \left\{\begin{array}{c} \underline{u}_t \\ \hline \underline{u}_t^b \end{array}\right\} = \underline{R}(t) \tag{71}$$

Substituting Eq. (69) into Eq. (71), one obtains

$$\underline{M}\ \ddot{\underline{u}} + \underline{C}_t\ \dot{\underline{u}} + \underline{K}_t\ \underline{u} = \underline{R}(t) - \left[\underline{M}\ \underline{M}^b\right] \left\{\begin{array}{c} \ddot{\underline{u}}_s \\ \ddot{\underline{u}}_s^b \end{array}\right\} - \left[\underline{C}_t\ \underline{C}_t^b\right] \left\{\begin{array}{c} \dot{\underline{u}}_s \\ \dot{\underline{u}}_s^b \end{array}\right\} \tag{72}$$

after making use of the relation

$$\underline{K}_t\ \underline{u}_s + \underline{K}_t^b\ \underline{u}_s^b = \underline{0}\ , \tag{73}$$

which is satisfied by definition of the quasi-static vector $\underline{u}_s$. This vector can thus be obtained directly from Eq. (73), i.e.,

$$\underline{u}_s = -\underline{K}_t^{-1}\ \underline{K}_t^b\ \underline{u}_s^b \equiv -\underline{B}_t\ \underline{u}_s^b \tag{74}$$

where $\underline{B}_t \equiv \underline{K}_t^{-1}\ \underline{K}_t^b$ is a matrix of quasi-static influence coefficients resulting from the n^b non-zero support displacements. If the system

is linear, all coefficients in matrix $\underline{B}_t$ are invariant with time.

Usually the damping terms on the right hand side of Eq. (72) are small compared to the inertia terms and therefore may be dropped from the equation without introducing significant errors. Also, the coefficients in matrix $\underline{M}^b$ can be set equal to zero since mass coupling vanishes for a lumped mass model. Thus, after substituting Eq. (74) into Eq. (72), the equations of motion reduce to the form

$$\underline{M}\,\underline{\ddot{u}} + \underline{C}_t\,\underline{\dot{u}} + \underline{K}_t\,\underline{u} = \underline{R}(t) + \underline{M}\,\underline{B}_t\,\underline{\ddot{u}}_s^b \tag{75}$$

where $\underline{\ddot{u}}_s^b$ is a vector containing the prescribed support excitations.

1. Multiple Support Excitations - When ground excitations corresponding to each of the n^b support displacements are prescribed by a vector $\underline{\ddot{u}}_g^m(t)$, the vector $\underline{\ddot{u}}_s^b$ in Eq. (75) can be expressed as

$$\underline{\ddot{u}}_s^b = \underline{\ddot{u}}_g^m(t) \tag{76}$$

and the equations of motion become

$$\underline{M}\,\underline{\ddot{u}} + \underline{C}_t\,\underline{\dot{u}} + \underline{K}_t\,\underline{u} = \underline{R}(t) + \underline{M}\,\underline{B}_t\,\underline{\ddot{u}}_g^m(t) \tag{77}$$

2. Rigid Support Excitations - When ground excitations at all support points along the base of the structure are identical and are prescribed by a rigid ground acceleration vector $\underline{\ddot{u}}_g^r$ consisting of three translational components $\ddot{u}_{gX}$, $\ddot{u}_{gY}$, and $\ddot{u}_{gZ}$, measured along their corresponding global axes X, Y, and Z, i.e.,

$$\underline{\ddot{u}}_g^r = \begin{Bmatrix} \ddot{u}_{gX} \\ \ddot{u}_{gY} \\ \ddot{u}_{gZ} \end{Bmatrix} \tag{78}$$

the equations of motion become

$$\underline{M}\,\underline{\ddot{u}} + \underline{C}_t\,\underline{\dot{u}} + \underline{K}_t\,\underline{u} = \underline{R}(t) + \underline{M}\,\underline{B}^r\,\underline{\ddot{u}}_g^r(t) \tag{79}$$

reliable phase difference predictions can be made. Therefore, ground motions in the form of rigid base motions are used in analyses.

C. LINEAR ANALYSIS PROCEDURES - MODE SUPERPOSITION METHOD

When the structural system is linear, the stiffness and damping matrices are invariant with time, i.e., $\underline{K}_t = \underline{K}$ and $\underline{C}_t = \underline{C}$. Thus Eq. (81) takes on the linear form

$$\underline{M}\,\ddot{\underline{u}} + \underline{C}\,\dot{\underline{u}} + \underline{K}\,\underline{u} = \underline{R}(t) + \underline{M}\,\underline{B}\,\ddot{\underline{u}}_g(t) \tag{82}$$

A standard procedure can be followed in obtaining dynamic response, namely, solving the generalized eigenvalue problem for mode shapes and frequencies, solving a decoupled set of normal equations of motion, and using mode superposition to obtain time histories of response [18].

1. Mode Shapes and Frequencies - The desired undamped free vibration mode shapes and corresponding frequencies can be obtained by solving the equation

$$\underline{K}\,\underline{\phi}_i = \omega_i^2\,\underline{M}\,\underline{\phi}_i \qquad (i=1,2,\text{---},q);\ q \leq n \tag{83}$$

where ω_i and $\underline{\phi}_i$ are the frequency and shape vector, respectively, for the ith mode and where q is the number of lowest modes required for the accuracy of solution desired. Usually, q is much less than n and can be efficiently determined using a determinant search technique combined with inverse iteration or a subspace iteration technique as recently developed by Bathe [26].

The modal matrix $\underline{\Phi} \equiv [\underline{\phi}_1\ \underline{\phi}_2 \text{---}\ \underline{\phi}_q]$ must satisfy the orthogonality condition

$$\underline{\Phi}^T\,\underline{K}\,\underline{\Phi} = \underline{\Omega}^2 \tag{84}$$

where $\underline{\Omega}^2$ is a diagonal matrix containing the squared frequencies $\omega_1^2, \omega_2^2, ---, \omega_q^2$. It is convenient to normalize the modal matrix so that it satisfies the condition

$$\underline{\Phi}^T \underline{M}\, \underline{\Phi} = \underline{I} \tag{85}$$

where $\underline{I}$ is the unit (or identity) matrix.

Vectors $\underline{u}(t)$, $\underline{\dot{u}}(t)$, and $\underline{\ddot{u}}(t)$ can be expressed in terms of the modal matrix and the normal coordinate (mode amplitude) vector $\underline{U}(t)$ as follows:

$$\underline{u} = \underline{\Phi}\, \underline{U} \; ; \; \underline{\dot{u}} = \underline{\Phi}\, \underline{\dot{U}} \; ; \; \underline{\ddot{u}} = \underline{\Phi}\, \underline{\ddot{U}} \tag{86}$$

Substituting Eq. (86) into Eq. (82), premultiplying the resulting equation by $\underline{\Phi}^T$, and making use of Eq. (84) and (85), one obtains

$$\underline{\ddot{U}} + \underline{\Lambda}\, \underline{U} + \underline{\Omega}^2\, \underline{U} = \underline{R}^*(t) \tag{87}$$

where $\underline{\Lambda}$ is a diagonal matrix containing the terms $2\xi_i\omega_i$, for $i=1,2,---,q$ and where $\underline{R}^*(t)$ is a normal load vector defined by

$$\underline{R}^*(t) = \underline{\Phi}^T \left[\underline{R}(t) + \underline{M}\, \underline{B}\, \underline{\ddot{u}}_g(t) \right] \tag{88}$$

Using Eq. (6), matrix $\underline{\Lambda}$ can be expressed in the form

$$\underline{\Lambda} = \underline{\Phi}^T \underline{C}\, \underline{\Phi} = \underline{\Phi}^T [\, \alpha\underline{M} + \beta\underline{K}\,]\, \underline{\Phi} = \alpha\underline{I} + \beta\underline{\Omega}^2 \tag{89}$$

2. Response Time Histories - The solution of Eq. (87) can be carried out in the time domain using the convolution integral [18]

$$U_i(t) = \frac{1}{\omega_{Di}} \int_o^t R_i^*(\tau)\, e^{-\xi_i\omega_i(t-\tau)} \sin \omega_{Di}(t-\tau)\, d\tau \tag{90}$$

for $i=1,2,---,q$;

where ω_{Di} is the damped frequency

$$\omega_{Di} = \omega_i (1-\xi_i^2)^{\frac{1}{2}} \tag{91}$$

Thus, the response time histories $\underline{u}(t)$, $\dot{\underline{u}}(t)$, and $\ddot{\underline{u}}(t)$ can now be obtained using Eq. (86).

3. Spectral Responses - For a given single component of earthquake ground acceleration $\ddot{u}_g(t)$, the displacement response spectra $S_d(\omega,\xi)$ define the maximum (or extreme) values of displacement governed by the equation [27]

$$\ddot{\upsilon} + 2\xi\omega\ \dot{\upsilon} + \omega^2\ \upsilon = \ddot{u}_g(t) \tag{92}$$

These spectra are usually obtained directly from the pseudo-velocity response spectra as defined by

$$S_d(\omega,\xi) \equiv \frac{S_v(\omega,\xi)}{\omega} \tag{93}$$

Suppose these response spectra are known for each ground acceleration input in the vector $\ddot{\underline{u}}_g(t)$ appearing in Eq. (88) and suppose the $\underline{R}(t) = \underline{0}$, then the modal maximum (or spectral) response can be obtained from the relation

$$|\ \upsilon_i(t)\ |_{max.} = \underline{\phi}_i^T\ \underline{M}\ \underline{B}\ \underline{S}_d(\omega_i,\xi_i) \qquad (i=1,2,---,q) \tag{94}$$

The maximum displacement response $|\underline{u}_i(t)|_{max.}$ can then be determined using

$$|\ \underline{u}_i(t)\ |_{max.} = \underline{\phi}_i\ |\upsilon_i(t)|_{max.} \qquad (i=1,2,---,q) \tag{95}$$

Thus, the root-mean-square spectral displacement response can be expressed as

$$\left| \underline{u}(t) \right|_{R.M.S.} \equiv \left\{ \sum_{i=1}^{q} \left| \underline{u}_i(t) \right|^2_{max.} \right\}^{\frac{1}{2}} \tag{96}$$

D. NONLINEAR ANALYSIS PROCEDURE - STEP-BY-STEP INTEGRATION

When the structural system is nonlinear, the coupled equations of motion, Eq. (81), must be solved by step-by-step integration methods since mode superposition no longer applies. To carry out this integration, the equations of motion have been formulated on an incremental basis. Considering a time interval Δt starting at time t and assuming the stiffness and damping matrices at time t, i.e., $\underline{K}_t$ and $\underline{C}_t$, can be applied over the full time interval, one obtains the equations of motion in the incremental form

$$\underline{M}\ \Delta\ddot{\underline{u}} + \underline{C}_t\ \Delta\dot{\underline{u}} + \underline{K}_t\ \Delta\underline{u} = \Delta\ \underline{R}(t) + \underline{M}\ \underline{B}\ \Delta\ddot{\underline{u}}_g(t) \tag{97}$$

where

$$\begin{aligned} \Delta\ddot{\underline{u}} &= \Delta\ddot{\underline{u}}(t) = \ddot{\underline{u}}(t+\Delta t) - \ddot{\underline{u}}(t) \\ \Delta\dot{\underline{u}} &= \Delta\dot{\underline{u}}(t) = \dot{\underline{u}}(t+\Delta t) - \dot{\underline{u}}(t) \\ \Delta\underline{u} &= \Delta\underline{u}(t) = \underline{u}(t+\Delta t) - \underline{u}(t) \end{aligned} \tag{98}$$

and where

$$\begin{aligned} \Delta\underline{R}(t) &= \underline{R}(t+\Delta t) - \underline{R}(t) \\ \Delta\ddot{\underline{u}}_g(t) &= \ddot{\underline{u}}_g(t+\Delta t) - \ddot{\underline{u}}_g(t) \end{aligned} \tag{99}$$

To carry out the step-by-step numerical integration of Eq. (97), it is necessary to introduce an approximate operator in the time domain to replace the differential operator. In the present analysis, the Newmark generalized acceleration method [28] is adopted which

assumes the following approximations for nodal velocities and displacements:

$$\begin{aligned} \dot{\underline{u}}(t+\Delta t) &= \dot{\underline{u}}(t) + [(1-\delta)\,\ddot{\underline{u}}(t) + \delta\,\ddot{\underline{u}}(t+\Delta t)] \\ \underline{u}(t+\Delta t) &= \underline{u}(t) + \dot{\underline{u}}(t)\,\Delta t + [(\tfrac{1}{2}-\sigma)\ddot{\underline{u}}(t) + \sigma\,\ddot{\underline{u}}(t+\Delta t)] \end{aligned} \tag{100}$$

where parameters δ and σ can be chosen to give the required integration stability and accuracy. When $\delta = \frac{1}{2}$ and $\sigma = \frac{1}{6}$, the approximations correspond to the linear acceleration method, and when $\delta = \frac{1}{2}$ and $\sigma = \frac{1}{4}$ they correspond to the constant acceleration method. While the linear acceleration method is conditionally stable depending upon the magnitude of Δt, the constant acceleration method is unconditionally stable for any magnitude of Δt [29,30].

Using Eq. (97), the approximations given by Eq. (100) can be expressed in the incremental form

$$\begin{aligned} \Delta\ddot{\underline{u}}(t) &= C_1\,\Delta\underline{u}(t) - C_3\,\dot{\underline{u}}(t) - C_4\,\ddot{\underline{u}}(t) \\ \Delta\dot{\underline{u}}(t) &= C_2\,\Delta\underline{u}(t) - C_4\,\dot{\underline{u}}(t) - C_5\,\ddot{\underline{u}}(t) \end{aligned} \tag{101}$$

where

$$\begin{aligned} &C_1 = \frac{4}{\Delta t^2}, \quad C_2 = \frac{2}{\Delta t}, \quad C_3 = \frac{4}{\Delta t} \\ &C_4 = 2, \quad C_5 = 0 \;; \end{aligned} \tag{102}$$

for the constant acceleration method and where

$$\begin{aligned} &C_1 = \frac{6}{\Delta t^2}, \quad C_2 = \frac{3}{\Delta t}, \quad C_3 = \frac{6}{\Delta t} \\ &C_4 = 3, \quad C_5 = \frac{\Delta t}{2}, \end{aligned} \tag{103}$$

for the linear acceleration method.

Substituting Eq. (101) into Eq. (97), one obtains after some manipulation

$$\left[C_1 \underline{M} + C_2 \underline{C}_t + \underline{K}_t\right] \Delta\underline{u} = \Delta\underline{\bar{\bar{R}}}(t) \tag{104}$$

where

$$\begin{aligned}\Delta\underline{\bar{\bar{R}}}(t) = \Delta\underline{R}(t) + \underline{M}\,\underline{B}\,\Delta\underline{\ddot{u}}_g(t) &+ \left\{C_3\underline{M} + C_4\underline{C}_t\right\}\underline{\dot{u}}(t) \\ &+ \left\{C_4\underline{M} + C_5\underline{C}_t\right\}\underline{\ddot{u}}(t)\end{aligned} \tag{105}$$

Using Eq. (8), i.e., $\underline{C}_t = \alpha\underline{M} + \beta\underline{K}_t$, and introducing the following normalized constants

$$\begin{aligned} C_7 &= 1/(1 + \beta C_2) \quad , \quad & C_8 &= C_7\ (C_1 + \alpha C_2) \\ C_9 &= C_7\ (C_3 + \alpha C_4) \quad , \quad & C_{10} &= \beta C_4 C_7 \\ C_{11} &= C_7\ (C_4 + \alpha C_5) \quad , \quad & C_{12} &= \beta C_5 C_7 \end{aligned} \tag{106}$$

Eq. (104) can be put into the form

$$\begin{aligned}\left\{C_8\underline{M} + \underline{K}_t\right\}\Delta\underline{u} = C_7 &\left\{\Delta\underline{R}(t) + \underline{M}\,\underline{B}\,\Delta\underline{\ddot{u}}_g(t)\right\} \\ &+ \left\{C_9\,\underline{M} + C_{10}\underline{K}_t\right\}\underline{\dot{u}}(t) + \left\{C_{11}\underline{M} + \underline{C}_{12}\underline{K}_t\right\}\underline{\ddot{u}}(t)\end{aligned} \tag{107}$$

Defining an effective displacement vector $\Delta\underline{\bar{u}}(t)$ by the relation

$$\Delta\underline{\bar{u}}(t) = \Delta\underline{u}(t) - C_{10}\,\underline{\dot{u}}(t) - C_{12}\,\underline{\ddot{u}}(t) \tag{108}$$

and defining two new constants

$$\begin{aligned} C_{13} &\equiv C_9 - C_8\,C_{10} \\ C_{14} &\equiv C_{11} - C_8\,C_{12} \end{aligned} \tag{109}$$

Eq.(107) can be written in the form

$$\underline{\bar{K}}_t\,\Delta\underline{\bar{u}} = \Delta\underline{\bar{R}}(t) \tag{110}$$

where

$$\bar{\underline{K}}_t = C_8\, \underline{M} + \underline{K}_t \tag{111}$$

and

$$\Delta\bar{\underline{R}}(t) = C_7 \left\{ \Delta\underline{R}(t) + \underline{M}\,\underline{B}\,\ddot{\underline{u}}_g(t) \right\} + C_{13}\,\underline{M}\,\dot{\underline{u}}(t) + C_{14}\,\underline{M}\,\ddot{\underline{u}}(t) \tag{112}$$

Matrix $\bar{\underline{K}}_t$ is the effective dynamic stiffness matrix and $\Delta\bar{\underline{R}}(t)$ is the effective load vector.

Equation (110) can be solved for $\Delta\bar{\underline{u}}(t)$ at each time instant t and Eqs. (108) and (101) can be used to obtain $\Delta\underline{u}(t)$, $\Delta\dot{\underline{u}}(t)$, and $\Delta\ddot{\underline{u}}(t)$. The displacements, velocities, and accelerations at time $t + \Delta t$ can then be determined from Eq. (98).

The displacement solutions $\underline{u}(t + \Delta t)$ can be used to calculate the internal force vector $\underline{S}(t + \Delta t)$ and the new tangent stiffness matrix $\underline{k}_{t + \Delta t}$ for each nonlinear element in the system. The new total tangent stiffness matrix $\underline{K}_{t + \Delta t}$ is then obtained by the standard assemblage procedure. It is important to note that when determining the element internal forces, the static forces existing in the element must always be included since the calculation of element tangent stiffnesses depend upon the magnitudes of the element forces.

The step-by-step integration algorithm for the nonlinear system presented above may be summarized as follows:

(1) Initial calculations

- (a) Form the initial stiffness matrix $\underline{K}$ and the mass matrix $\underline{M}$ for the system.
- (b) Solve for the initial displacements and the element forces due to static loads.
- (c) Compute the initial quasi-static influence matrix $\underline{B}$, if required.

(d) Set up the dynamic load and ground excitation time histories $\underline{R}(t)$ and $\ddot{\underline{u}}_g(t)$.

(e) Calculate the step-by-step integration constants, C_i, $i=1,2,---,14$.

(2) For each time increment Δt at time t

(a) Form the tangent stiffness matrix $\underline{K}_t$.

(b) Compute the current quasi-static influence matrix $\underline{B}_t$, if required.

(c) Form the effective dynamic stiffness matrix $\bar{\underline{K}}_t$.

(d) Triangularize $\bar{\underline{K}}_t$.

(e) Form the effective load vector $\Delta\bar{\underline{R}}(t)$.

(f) Solve for the effective displacement increments $\Delta\bar{\underline{u}}(t)$

(g) Compute current accelerations, velocities, and displacements $\ddot{\underline{u}}(t)$, $\dot{\underline{u}}(t)$, $\underline{u}(t)$.

(h) Calculate current element forces, check nonlinearity conditions, and compute new element tangent stiffness matrices, inelastic deformation vectors, and unbalanced force vectors, if necessary.

(i) Return to step (a) or (e), as necessary, for next time increment.

VI LINEAR AND NONLINEAR SEISMIC ANALYSIS COMPUTER PROGRAMS

A. LINEAR ANALYSIS COMPUTER PROGRAM - BSAP

A computer program BSAP has been developed for dynamic response analysis of linear bridge structural systems. This program is a modified version of the SAP program originally developed by Wilson and co-workers for static and dynamic analyses of linear structural systems [31]. Although the original version of SAP is very efficient and can be used for analyzing a wide variety of structures, some modifications are necessary to make it suitable for the analysis of bridge structural systems. While most of BSAP follows the organization of SAP [31], additional development and modifications have been made on the SAP program as follows:

(1) New eigenvalue solution routines using a determinant search technique combined with inverse iteration or subspace iteration, both developed by Bathe [26], have been incorporated into the program for the determination of frequencies and mode shapes.

(2) In the dynamic response time history analysis option of the program, modifications have been made to permit multiple support excitations of the structural system. One may wish to specify this type of input for some bridge structures which are extremely long.

(3) A subroutine which optimizes the block storage of the stiffness and mass matrices of the system on the basis of available core storage and analysis type has been incorporated into the program.

(4) A linear elastic circularly curved beam element has been developed and added to the element library for modelling curved decks.

(5) A three-dimensional boundary spring element has been added to the element library for modelling foundation stiffnesses.

(6) A linear expansion joint element has been developed and added to the element library for modelling joint behavior.

Thus, the present version of BSAP can be used to analyze linear bridge structural systems modelled by any combination of the following element types: (1) three-dimensional truss element, (2) three-dimensional straight beam element, (3) three-dimensional curved beam element, (4) three-dimensional boundary spring element, and (5) linear expansion joint element. It can evaluate (1) response to static loadings, (2) frequencies and mode shapes, (3) time history responses to apply dynamic loadings and/or prescribed ground excitations (rigid or multiple), and (4) modal spectral responses and the root-mean-square values using prescribed earthquake spectra.

The organization of BSAP follows the linear analysis procedures established previously which can be described by the following operational sequence [31]:

(1) Nodal coördinates and the degrees of freedom of the system are generated from nodal input data. The nodal points may be specified as fully or partially constrained in any of its displacement coordinates. Nodes at multiple support input points are identified separately.

(2) Element data are input; then, matrices of stiffness, mass, and local-to-global coordinate transformation are generated and placed in low-speed storage units, e.g., disc files.

(3) Total stiffness and mass matrices for the complete system are assembled in one or more core blocks according to element connection arrays. Load vectors are formed in

the case of static analysis and the support coupling stiffness matrix $\underline{K}^b$ is assembled in the case of time history response analysis when considering multiple support excitations.

(4) Equilibrium equations are solved for static response or for quasi-static influence matrix $\underline{B}$ when prescribing multiple support excitations.

(5) For dynamic analysis, a specified number of lowest frequencies and mode shapes are determined.

(6) For time history response analysis, applied dynamic load and/or ground excitation time functions are input and digitized at equal time intervals. The response time histories are calculated using mode superposition procedure as described previously.

(7) For spectral response analysis, earthquake response spectra are input and modal spectral responses, as well as the root-mean-square responses, are determined based on prescribed modal damping ratios and the computed vibration frequencies.

Computer program BSAP is coded in FORTRAN IV. All storages are allocated in terms of variable dimensions at the time of execution, and an out-core block-by-block solution technique is adopted. Therefore, the program is capable of analyzing large bridge systems. It has been used to analyze several bridge structures using CDC 6600 and 7600 computers.

B. NONLINEAR ANALYSIS COMPUTER PROGRAM - NEABS

A new computer program NEABS has been developed for earthquake analysis of nonlinear bridge structural systems of the type considered

herein. This computer program uses the step-by-step integration procedures established in the previous chapter. Either the linear acceleration or constant acceleration method can be chosen for integration. The excitations can result from applied dynamic loads and/or from support motions (rigid or multiple). The program has an element library consisting of the five linear elements used in BSAP and nonlinear elements as follows: (1) three-dimensional elasto-plastic flexural column (straight) element for reinforced concrete members, (2) three-dimensional bilinear boundary spring element, and (3) nonlinear expansion joint element. In the program, a nonlinear element indicator has been used to allow a system modelled with both linear and nonlinear elements.

The organization of computer program NEABS can be described by the operational sequence:

(1) Static linear equilibrium equations are first generated and assembled in the same manner as in the linear analysis program BSAP. They are then solved for the static responses to be used as initial conditions in the subsequent nonlinear earthquake response analysis. In this same process, the initial quasi-static influence matrix is also determined, if needed.

(2) Dynamic loads and/or ground excitation time functions are input and are digitized for equal time intervals.

(3) Checks are then made on the available core storage to determine whether an in-core or an out-core block-by-block solution process is appropriate.

(4) For an in-core solution, an in-core step-by-step solution package is used to determine the response time histories of the system based on the procedures described previously.

(5) If an out-core block-by-block solution is necessary, an out-core step-by-step solution package can be used to evaluate the response time histories.

Computer program NEABS is written in FORTRAN IV and is coded in variable-dimension form. The program has been used to determine the nonlinear earthquake responses of a number of bridge structures using the CDC 6600 and 7600 computers.

Because of the extensive computational effort involved in a nonlinear earthquake response analysis, the out-core step-by-step solution package requires many slow-speed storage operations. Therefore, this procedure can be very costly and insufficient. For the bridge structures analyzed herein, the out-core solution has not been required.

The previously described mathematical modelling and analysis procedures have been employed to determine the response of over 10 bridge structural systems using computer programs BSAP and NEABS. These systems are basically patterned after 3 prototype bridge structures but with variations in certain parameters allowed. The 3 prototype structures are the 5/14 South Connector Overcrossing of the Golden State freeway and Antelope Valley freeway interchange, the curved Figueroa Street Undercrossing Connector in the Los Angeles area, and a straight version of the Figueroa Street Undercrossing Connector. The structural data and numerical results obtained for each bridge system are presented subsequently along with summaries of extreme-values of response.

A. 5/14 SOUTH CONNECTOR OVERCROSSING

1. The Structural System - The structural system of this bridge consists of a curved, continuous, reinforced concrete box girder deck of 9 spans (Nos. 1-9 from left to right in Fig. 14) supported on 8 single, central, reinforced concrete columns (Nos. 2-9 in Fig. 14), and 2 reinforced concrete diaphragm abutments (Nos. 1 and 10 in Fig. 14). The deck has a radius of curvature of 667 ft. measured to its center line. Four expansion joints (1-4 from left to right in Fig. 14) divide the deck into 5 continuous segments. The structural layout of this bridge system is shown in Fig. 14 and the structural properties of the bridge components are given in Table 1. During the San Fernando earthquake of February 9, 1971, this bridge suffered severe damage. Central spans Nos. 3 and 4 collapsed along with column No. 4.

For purpose of analysis, this bridge has been idealized by a discrete-parameter system consisting of 71 nodal points of which the 10 located at the bases of the 8 columns and the 2 abutments are fixed to the ground; thus, a total of 366 (61x6) degrees of freedom are

present in the system (Fig. 15). For linear analysis, a system of 28 three-dimensional linear straight beam elements, 38 three-dimensional linear curved beam elements, and 4 linear expansion joint elements have been used to model the structure as shown in Fig. 16.

For nonlinear analysis, linear elastic straight beam elements Nos. 3-28 as shown in Fig. 16 are replaced by 26 corresponding three-dimensional elasto-plastic flexural column elements and the 4 linear expansion joint elements are replaced by nonlinear elements.

The ultimate column strengths as computed by program YIELD are given in Table 2. The corresponding generalized yield function constants are shown in Table 3.

For nonlinear analysis, three different systems of longitudinal restrainers have been considered for the expansion joints (1) a strong system consisting of 3 short tie bars each having a $2\frac{1}{4}$ in. diameter, a 4ft. length, and a yield strength of 480 kips, (2) a weak system consisting of 3 short tie bars each having a $1\frac{1}{2}$ in. diameter, a 4ft. length, and a yield strength of 70.8 kips, and (3) a system consisting of no ties. The longitudinal tie bars are located across each expansion joint at the centers of the 3 box girder cells which are 9ft. apart. Prior to the San Fernando earthquake, longitudinal restrainer system No. 2 was generally used. However following this earthquake, restrainer system No. 1 has been proposed [32].

In all nonlinear cases studied, the vertical restrainers of the expansion joints were assumed to have infinite yield strengths, the Coulomb coefficients of friction were set at 0.4, the expansion joint gaps Δ_G were taken equal to 1 in., and the tie gaps were assumed equal to zero.

Foundation flexibilities are approximated in all cases by assuming the columns and abutments to be fully fixed at a depth 10ft. below the ground profile.

2. Mode Shapes and Frequencies - Frequencies and corresponding mode shapes were determined for two structural cases using program

BSAP. In the first case, zero friction was assumed for the expansion joints and, in the second case, infinite friction was assumed. The computed frequencies and corresponding periods for the lowest 10 modes of vibration are shown in Table 4 for the zero friction case and in Table 5 for the infinite friction case. The shapes of the 4 lowest modes are plotted in Figs. 17 and 18 for the zero and infinite friction cases, respectively.

3. Spectral Responses - Spectral responses of the linear bridge system subjected to the N-S El Centro 1940 earthquake motions in its transverse direction, i.e., the global Y direction in Fig. 15 were determined for both the zero and infinite friction cases using program BSAP. A damping ratio of 0.02 was used for all modes in these analyses.

Spectral responses for maximum bending moments about the local z axes in columns Nos. 4, 5, and 7 are shown for the lowest 10 modes in tables 6 and 7 for the zero and infinite friction cases, respectively. Root-mean-square values for all 10 modes are also shown in these tables.

4. Nonlinear Earthquake Responses - Nonlinear earthquake response analyses using computer program NEABS were carried out for five structural cases. In all cases, the ground excitation inputs were derived from an artificial earthquake record (OVH accelerogram) generated by Chopra [33] as shown in Fig. 19. This accelerogram was generated to simulate the ground motions produced by the San Fernando earthquake at the site of the Olive View Hospital located about 6 miles southwest of the epicenter. It has a peak acceleration of 0.5g, a uniform phase of high intensity shaking for 8 seconds, and a spectrum intensity of 4.01 ft. which is approximately 1.5 times greater than the N-S El Centro 1940 earthquake.

For all five nonlinear cases studied, rigid ground excitations were assumed and the intensity levels, i.e., peak accelerations, were adjusted to those values shown in Table 8. The structural parameter varied in these five cases was a type of longitudinal restrainer system.

The numerical results obtained for these five cases are based on damping constants $\alpha = 0.0419$ and $\beta = 0.0079$ which correspond to damping ratios of 0.02 in the first two transverse modes of vibration for the zero friction linear case as shown in Fig. 17. In all cases, a time interval of 0.02 seconds and the constant acceleration method have been used in step-by-step integration. Representative time histories of response for these five cases are presented as follows:

<u>Case 1</u> - The horizontal accelerations (absolute) and the displacements in the global X and Y directions at the top of columns Nos. 3, 4, and 5 are shown in Figs. 20, 21, and 22, respectively. The vertical accelerations and displacements at the center of spans Nos. 3 and 4 are given in Fig. 23. The generalized forces at the center of span No. 3 are shown in Fig. 24. The bending moments and corresponding plastic bending rotations at the bases of columns Nos. 3, 4, 5, and 6 are presented in Figs. 25, 26, 27, and 28, respectively. Longitudinal joint separations at expansion joints Nos. 1 and 2 are shown in Fig. 29. The forces and corresponding plastic elongations for the 3 longitudinal tie bars at expansion joints Nos. 1 and 2 are given in Figs. 30 and 31, respectively. The forces in the vertical restrainers of expansion joints Nos. 1 and 2 are presented in Fig. 32.

<u>Case 2</u> - The horizontal accelerations and displacements at the top of columns Nos. 3, 4, and 5 are shown in Figs. 33, 34, and 35, respectively. The vertical accelerations and displacements at the centers of spans Nos. 3 and 4 are given in Fig. 36. The generalized forces at the center of span No. 4 are shown in Fig. 37. The bending moments and corresponding plastic bending rotations at the bases of columns Nos. 3, 4, 5, and 6 are presented in Figs. 38, 39, 40, and 41, respectively. The longitudinal joint separations at expansion joints Nos. 1 and 2 are shown in Fig. 42. The longitudinal tie bar forces and corresponding plastic elongations at expansion joints Nos. 1

and 2 are shown in Figs. 43 and 44, respectively. Vertical restrainer forces at expansion joints Nos. 1 and 2 are given in Fig. 45.

Case 3 - This particular bridge system experienced excessively large amplitudes of response. For example, note the large horizontal displacements which occurred at the top of column No. 4 as shown in Fig. 46 and the large joint separations which developed at expansion joint No. 2 as presented in Fig. 47. Due to the fact that very large geometry changes resulted in this case, the numerical results obtained are not valid. They do however strongly indicate that some portions of the overall structure would collapse.

Case 4 - The horizontal accelerations and displacements at the top of column No. 4 are shown in Fig. 48. The vertical accelerations and displacements at the top of this same column and at the center of span No. 4 are presented in Fig. 49. The generalized forces in the box girder at the center of span No. 4 are given in Fig. 50. The bending moments and corresponding plastic bending rotations at the bases of columns No. 3, 4, and 5 are shown in Figs. 51, 52, and 53, respectively. The longitudinal joint separations at expansion joint No. 2 are presented in Fig. 54. The longitudinal tie bar forces and corresponding plastic elongations at expansion joint No. 2 are given in Fig. 55.

Case 5 - The horizontal accelerations and displacements at the top of column No. 4 are given in Fig. 56 while the vertical accelerations and displacements at the top of this same column and at the center of span No. 4 are presented in Fig. 57. The generalized forces in the box girder at the center of span No. 4 are shown in Fig. 58. The bending moments and corresponding plastic bending rotations at the bases of columns Nos. 3, 4, and 5 are presented in Figs. 59, 60, and 61, respectively. The longitudinal joint separations at expansion joint No. 2 as

shown in Fig. 62 and the longitudinal tie bar forces and corresponding plastic elongations are shown in Fig. 63.

B. CURVED FIGUEROA STREET UNDERCROSSING CONNECTOR

1. The Structural System - The structural system of this bridge consists of a curved, continuous, reinforced concrete box girder deck of 6 spans (Nos. 1-6 from left to right in Fig. 64) supported on 5 reinforced concrete columns (Nos. 2-6 in Fig. 64) and 2 reinforced concrete diaphragm abutments (Nos. 1 and 7 in Fig. 64). The deck has a radius of curvature of 886 ft. measured to its center line. Only one expansion joint is present (No. 1). All 5 columns have a length of 40ft. and are flared at their upper ends as shown in Fig. 65. These columns are rigidly connected at their bases to massive reinforced concrete footings (25' x 25' x 6') which are in turn supported on 36 piles each. Therefore, the foundation stiffnesses are sufficiently high so that the columns can be assumed as rigidly fixed at their bases. The box girders are rigidly connected to the diaphragm abutments which are relatively short (6 ft.). These abutments are supported on spread footings which provide high shear resistances but low bending resistances. The structural properties of this bridge system are summarized in Table 9.

The discrete parameter system used to model this bridge is shown in Fig. 66. It contains 62 nodal points of which 5 are located at the fixed bases of columns and 2 are located at partially fixed bases which provide full constraint in the three translational directions and in the rotational direction about a longitudinal axis parallel to the deck; however, no significant degree of fixity is provided in the other two rotational directions. Considering these support constraints, the overall system has 274 degrees of freedom.

For linear analysis, a finite element system consisting of 27 three-dimensional straight beam elements, 23 three-dimensional curved beam elements, and 1 linear expansion joint element as shown in Fig. 67 was used. Each column was modelled by 5 straight beam elements of which 3 were used to model the upper flared section

and 2 were used to model the prismatic portion. For nonlinear analysis, the 2 straight beam elements of the prismatic section were replaced by 2 corresponding elasto-plastic flexural column elements. The ultimate strengths of these elements as calculated by YIELD are shown in Table 10. The corresponding generalized yield function constants are given in Table 11.

The expansion joint of this bridge uses a longitudinal restrainer system consisting of 6 cable units, placed across the joint at distances of $\pm$ 8.92, $\pm$ 5.85, $\pm$ 2.92 ft. from the center line of the deck. Each cable unit consists of 5 - $1\frac{3}{4}$ in. diameter cables, 127.5 ft. long, having yield strengths of 154.6 kips each. The expansion joint is constructed with an initial joint gap Δ_G = 6 in. but with all tie gaps Δ_T equal to zero. The vertical restrainers at the expansion joint are assumed to have infinite yield strengths. The Coulomb friction coefficient is again assumed to be 0.4. The expansion joint width d is taken as 27ft.

The longitudinal restrainer system used in this bridge is a new version recently developed by the California State Division of Highways [32].

2. <u>Mode Shapes and Frequencies</u> - The mode shapes and frequencies of this bridge system have been determined only for the case of zero friction in the expansion joint. The computed frequencies and corresponding periods for the lowest 10 modes are listed in Table 12. The shapes of the lowest 4 modes are plotted in Fig. 68.

3. <u>Nonlinear Earthquake Responses</u> - The nonlinear response of this bridge system has been determined using NEABS for three different cases. In all three cases, rigid ground excitations corresponding to the OVH accelerogram were used as shown in Fig. 19. The structural parameter varied in these studies were the yield strengths of the columns. Both elastic and elasto-plastic flexural columns were considered. Table 13 describes the column types used in each case and also shows the intensities of excitations which were prescribed.

Numerical results have been obtained for all three cases using

damping constants $\alpha = 0.09$ and $\beta = 0.0012$ which correspond to a damping ratio of 0.02 for the lowest 2 modes of vibration for the zero friction case. In all 3 cases, a time interval of 0.02 seconds has been used in step-by-step integration. Selected time history responses are presented as follows:

Case 1 - The horizontal accelerations and displacements at the top of column No. 3 and 4 are presented in Figs. 69 and 70, respectively. The vertical accelerations and displacements at the top of column No. 3 and at the center of span No. 3 are shown in Fig. 71. The generalized forces in the box girder at the center of span No. 3 are given in Fig. 72 while the elastic bending moments at the bases of columns Nos. 3 and 4 are presented in Fig. 73. The longitudinal joint separations are shown in Fig. 74 and the longitudinal cable forces at the joint are given in Fig. 75. These cable forces remain elastic during the entire response history.

Case 2 - The horizontal accelerations and displacements at the top of columns Nos. 3 and 4 are shown in Figs. 76 and 77, respectively. The vertical accelerations and displacements at the top of column No. 3 and at the center of span No. 3 are given in Fig. 78. The generalized forces in the box girder at the center of span No. 3 are presented in Fig. 79 while the bending moments and corresponding plastic rotations at the bases of columns Nos. 3 and 4 are shown in Figs. 80 and 81, respectively. The longitudinal joint separations are given in Fig. 82 and the longitudinal cable forces are presented in Fig. 83. These cable forces also remain elastic during the entire response history.

Case 3 - The horizontal accelerations and displacements at the top of column No. 4 are shown in Fig. 84 while the vertical accelerations and displacements at the top of column No. 3 and at the center of span No. 3 are given in Fig. 85. The generalized forces in the box girder at the center of span No. 3 are presented in Fig. 86 while the bending moments and correspond-

ing plastic rotations at the bases of columns Nos. 3 and 4 are shown in Figs. 87 and 88, respectively. The longitudinal joint separations are presented in Fig. 89 and the longitudinal cable forces are given in Fig. 90. It is significant to note that the cables remained elastic throughout the time history of response.

C. STRAIGHT FIGUEROA STREET UNDERCROSSING CONNECTOR

1. The Structural System - The structural system of this bridge is identical to that of the curved Figueroa Street Undercrossing Connector described previously except that the curvature of the deck has been removed. The general layout of the bridge system is shown in Fig. 91 and the discrete parameter system selected for analysis is given in Fig. 92. This discrete parameter system has the same nodal point arrangement as that selected for the curved bridge model. For linear analysis, the structure has been modelled by 50 three-dimensional straight beam elements and 1 linear expansion joint as shown in Fig. 93. For nonlinear analysis, 2 three-dimensional elasto-plastic flexural elements have been used to model the lower 22ft. prismatic section of each column. The ultimate strengths and corresponding yield function constants for these columns are listed in Tables 10 and 11.

2. Mode Shapes and Frequencies - Mode shapes and frequencies of this bridge system have been determined for the zero friction case only. Frequencies and corresponding periods for the lowest 10 modes are listed in Table 14 while the corresponding shapes of the lowest 4 modes are shown in Fig. 94.

3. Nonlinear Earthquake Responses - Nonlinear response analyses for this system have been carried out using NEABS for only one case. The rigid ground excitations used corresponded with the OVH accelerograms. The intensities used were 0.5g in the transverse direction (global Y direction) and 0.3g in the vertical direction (global Z

direction).

The nonlinear analysis was carried out using damping constants $\alpha = 0.09$ and $\beta = 0.0013$ which correspond to a damping ratio of 0.02 for the lowest 2 modes of vibration in the zero friction case. A time interval of 0.02 seconds has also been used. The horizontal accelerations and displacements at the top of columns Nos. 3 and 4 are shown in Figs. 95 and 96, respectively, while the vertical accelerations and displacements at the top of column No. 3 and at the center of span No. 3 are given in Fig. 97. The generalized forces in the box girder at the center of span No. 3 are presented in Fig. 98 and the bending moments and corresponding plastic rotations at the base of columns Nos. 3 and 4 are shown in Figs. 99 and 100, respectively. The longitudinal joint separations are given in Fig. 101 and the longitudinal cable forces are shown in Fig. 102. It should be noted that the cable forces remained elastic throughout the time history of response.

D. SUMMARY OF EXTREME-VALUES OF NONLINEAR RESPONSES

The extreme-values of the responses previously presented for the 5 cases of the 5/14 South Connector Overcrossing, the 3 cases of the curved Figueroa Street Undercrossing Connector, and the single case of the straight Figueroa Street Undercrossing Connector are summarized as follows:

1. Maximum Accelerations and Displacements - The maximum horizontal accelerations (absolute) and displacements at the top of columns No. 2-7 are summarized in Tables 15 and 16, respectively, for the 5/14 South Connector Overcrossing. The maximum values shown are the vectorial sum of the horizontal components measured along the global X and Y axes. Likewise, the maximum horizontal accelerations and displacements at the top of each column for the 3 cases of the curved and the single case of the straight Figueroa Street Undercrossing Connector are shown in Tables 17 and 18, respectively.

The maximum vertical accelerations and displacements at the center of spans are shown in Tables 19 and 20, respectively, for the 5/14 South Connector Overcrossing and are shown in Tables 21 and 22, respectively, for the curved and straight Figueroa Street Undercrossing Connector.

2. Locations of Maximum Flexural Yielding in Columns - The locations of maximum flexural yielding in the columns of the 5/14 South Connector Overcrossing are shown in Fig. 103 for all 5 cases investigated. Similar locations for Cases 2 and 3 of curved Figueroa Street Undercrossing Connector and the single case of the straight Figueroa Street Undercrossing Connector are shown in Fig. 104. It should be noted that the hinge locations in these figures represent locations of maximum flexural yielding occurring during the first 10 seconds of response.

3. Maximum Local Bending Ductility Factors at the Bases of Columns - The maximum local bending ductility factor μ at a yield hinge of a column is defined as the ratio of the maximum flexural rotation to the yield rotation θ^Y, i.e.

$$\mu = \frac{\theta^P + \theta^Y}{\theta^Y} \tag{113}$$

where θ^P is the plastic component of flexural rotation. For a column of finite dimension, yielding will not occur in concentrated hinges; rather, it will occur over finite lengths h which are approximately equal to the transverse dimensions of the column cross-section in the directions normal to the axes of rotation [20]. Therefore, the flexural yield rotation θ^Y can be estimated by the relation

$$\theta^Y = \frac{M_u}{EI} h \tag{114}$$

where M_u is the ultimate bending moment which in the present analysis is assumed equal to the yield bending moment.

The flexural yield rotations θ_y^Y and θ_z^Y about the local y and z axes of the columns of the 5/14 South Connector Overcrossing have been computed using Eq. (114) and the results are listed in Table 23. Similar yield rotations for the columns of the Figueroa Street Undercrossing Connector are shown in Table 24.

The maximum local bending ductility factors at the bases of the columns of the 5/14 South Connector Overcrossing are computed using Eq. (113) for Cases 1, 2, 4, and 5. The results are shown in Table 25. Similar ductility factors for Cases 2 and 3 of the curved Figueroa Street Undercrossing Connector and the single case of the straight Figueroa Street Undercrossing Connector are given in Table 26. The values presented in Tables 25 and 26 are absolute maxima computed for the bending rotations about the local y and z axes.

4. Maximum Longitudinal Joint Separations and Longitudinal Restrainer Tie Ductility Factors - The maximum longitudinal expansion joint separations at the center lines of the bridge decks are shown in Table 27 for expansion joints Nos. 1-4 in Cases 1, 2, 4, and 5 of the 5/14 South Connector Overcrossing. Similar joint separations are shown in Table 28 for the single expansion joint of the Figueroa Street Undercrossing Connector (3 cases for the curved undercrossing and 1 case for the straight crossing).

The longitudinal restrainer tie ductility factor μ_T is defined as the ratio of the maximum tie elongation to its yield elongation, i.e.

$$\mu_T = \frac{u_T^E + u_T^P}{u_T^E} \tag{115}$$

where u_T^P and u_T^E are defined by Eq. (53) of Chapter IV. Factor μ_T is computed in each case for all longitudinal restrainer ties at the expansion joints. The maximum values of μ_T for each expansion joint are listed in Table 27 for 4 cases of the 5/14 South Connector Overcrossing and in Table 28 for 3 cases of the curved Figueroa Street Undercrossing Connector and for the single case of the straight

Figueroa Street Undercrossing Connector.

Table 1 Structural Properties of 5/14 South Connector Overcrossing

Structural Component	Member Dim. (ftXft)	Young's Modulus (k / ft^2)	Poisson Ratio	Unit Weight (lb/ft^3)	A (ft^2)	I_x (ft^4)	I_y (ft^4)	I_z (ft^4)
Column # 2	10 X 5	490,000	0.20	145.0	45.7	215.0	86.1	327.6
Column # 3.5	10 X 6	490,000	0.20	145.0	53.9	324.0	143.2	379.6
Column # 4	10 X 6	524,000	0.20	145.0	53.9	324.0	143.2	379.6
Column # 6	10 X 4	524,000	0.20	145.0	37.3	126.0	49.6	275.3
Column # 7.8	10 X 4	557,000	0.20	145.0	37.3	126.0	49.6	275.3
Column # 9	10 X 4	490,000	0.20	145.0	37.3	126.0	49.6	275.3
Abutment # 1	31 X 2.5	490,000	0.20	145.0	77.5	161.5	40.4	6200.0
Abutment # 10	34 X 2.5	490,000	0.20	145.0	85.0	177.2	44.3	8190.0
Girder @ Span # 1, 2, 5, 9	34 X 7	490,000	0.20	145.0	56.3	1107.6	479.3	5443.2
Girder @ Span #3, 4, 6, 7,8	34 X 7	524,000	0.20	145.0	60.9	1081.1	488.6	5722.0

Table 2 Ultimate Strength of the Columns of 5/14 South Connector Overcrossing

Column No.	f_c' (psi)	Reinforcement #18 bars	P_o 10^4 (k)	M_{yo} 10^4 (k-ft)	M_{zo} 10^4 (k-ft)
# 2	3,500	40	2.579	1.386	2.537
# 3, 5	3,500	40	2.948	1.671	2.598
# 4	4,000	52	3.470	2.156	3.331
# 6	4,000	40	2.465	1.102	2.517
# 7, 8	4,500	48	2.821	1.320	2.997
# 9	3,500	48	2.365	1.303	2.900

Table 3 Generalized Yield Function Constants of the Columns of 5/14 South Connector Overcrossing

Column No.	a	a_1	a_2	a_3	b	b_1	b_2	b_3
# 2	2.0	-3.10	-3.83	0.273	2.0	-2.97	-4.21	-0.244
# 3	2.0	-3.63	-4.53	0.102	2.0	-3.55	-4.80	-0.245
# 4	2.0	-3.20	-4.09	0.101	2.0	-3.12	-4.34	-0.225
# 6	2.0	-2.98	-3.44	0.546	2.0	-2.78	-4.00	-0.219
# 7, 8	2.0	-2.81	-3.32	0.489	2.0	-2.61	-3.85	-0.238
# 9	2.0	-2.22	-2.79	0.422	2.0	-2.01	-3.24	-0.232

Table 4 Frequencies and Periods of 5/14 South Connector Overcrossing - Zero Friction

Mode No.	Frequency (rad / sec)	Period (sec)
1	1.226	5.124
2	2.243	2.801
3	3.121	2.013
4	5.549	1.132
5	7.929	0.924
6	9.031	0.696
7	9.123	0.689
8	9.507	0.661
9	12.241	0.513
10	12.902	0.487

Table 5 Frequencies and Period of 5/14 South Connector Overcrossing - Infinite Friction

Mode No.	Frequency (rad / sec)	Period (sec)
1	4.500	1.391
2	6.209	1.012
3	9.109	0.690
4	9.585	0.655
5	11.301	0.556
6	13.487	0.466
7	14.262	0.441
8	14.668	0.428
9	15.123	0.415
10	16.013	0.392

Table 6 Spectral Responses of 5/14 South Connector Overcrossing to the N-S El Centro 1940 Earthquake —Zero Friction Case

Mode No.	Maximum Bending Moments in Columns * About the Local z Axis (k-ft)		
	Column # 4	Column # 5	Column # 7
1	22.5	102.7	1.5
2	29,311.0	31,334.0	2,115.4
3	2.1	398.7	30.4
4	1,635.1	7,828.6	2,455.0
5	639.7	2,359.4	17,458.0
6	902.3	5,519.3	12,080.0
7	40.0	18.5	16.9
8	747.9	762.2	82.9
9	437.8	697.4	1,664.9
10	1,841.6	791.3	94.2
R.M.S.	29,448.0	32,879.0	21,540.0

* Maximum moments shown all occur at the base of each column

Table 7 Spectral Responses of 5/14 South Connector Overcrossing to the N-S El Centro 1940 Earthquake — Infinite Friction Case

Mode No.	Maximum Bending Moments in Columns ** about the Local z Axis (k-ft)		
	Column # 4	Column # 5	Column # 7
1	554.2	480.7 *	884.6
2	11,573.0	10,141.0	1,928.9 *
3	41.8 *	446.9 *	237.5
4	2,136.2	4,358.1 *	12,570.0
5	5.2	5.8	4.5
6	23.6	95.5	20.9 *
7	10.0	1.4	164.9 *
8	237.1	12.8	745.4
9	100.5	79.8 *	293.3
10	61.8	38.9	222.5
R.M.S.	11,785.0	10,574.0	12,684.0

* Maximum moment occurs at the top of the column.

** All valves shown are maxima at the base of the columns, except those marked by *.

Table 8 Parameter Studies of 5/14 South Connector Overcrossing

Parameter / Case No.	Ground Excitation (OVH Ground Acceleration)		Longitudinal Restrainers at Expansion Joints
Case 1	Severe Shaking	0.5g Transverse (Y) 0.3g Vertical (Z)	3 Strong Short Ties *
Case 2	Severe Shaking	0.5g Transverse (Y) 0.3g Vertical (Z)	3 Weak Short Ties **
Case 3	Severe Shaking	0.5g Transverse (Y) 0.3g Vertical (Z)	No Ties
Case 4	Moderate Shaking	0.3g Transverse (Y) 0.18g Vertical (Z)	3 Weak Short Ties
Case 5	Severe Shaking	0.5g Transverse (Y)	3 Strong Short Ties

* Each strong short tie is $2\frac{1}{4}$" diameter steel bar at 4ft. long, S_y=480 kips.
** Each weak short tie is $1\frac{1}{2}$" diameter steel bar at 4ft. long, S_y=70.8 kips.

Table 9 Structural Properties of Figueroa Street Undercrossing Connector

Structural Component	Member Dim. (ft×ft)	Young's Modulus (k/ft^2)	Poisson's Ration	Unit Weight (lb/ft^3)	A (ft^2)	I_x (ft^4)	I_y (ft^4)	I_z (ft^4)
Column #2 *	20.0×7.5 7.5×7.5	432,000	0.18	145.0	114.2 45.6	1,728.0 295.0	758.6 163.5	1,436.0 163.5
Column #3,4*	20.0×7.5 7.5×7.5	432,000	0.18	145.0	114.2 45.6	1,728.0 295.0	758.6 163.5	1,436.0 163.5
Column #5 *	20.0×7.5 7.5×7.5	432,000	0.18	145.0	114.2 45.6	1,728.0 295.0	758.6 163.5	1,436.0 163.5
Column #6 *	20.0×7.5 7.5×7.5	432,000	0.18	145.0	114.2 45.6	1,728.0 295.0	758.6 163.5	1,436.0 163.5
Girder @ All Spans	40.5×7.5	432,000	0.18	145.0	81.2	1,340.0	587.4	9,606.0

* Also see Fig. 65.

Table 10 Ultimate Strength of the Columns of Figueroa Street Undercrossing Connector

Column No.	f_c' (lb/in^2)	Long. Bars #11	P_o 10^4 (k)	M_{yo} 10^4 (k-ft)	M_{zo} 10^4 (k-ft)
# 2	3,250	96	2.698	2.456	2.456
# 3,4	3,250	87	2.614	2.253	2.253
# 5	3,250	75	2.502	1.978	1.978
# 6	3,250	100	3.189	3.555	3.555

Table 11 Yield Function Constants of the Columns of Figueroa Street Undercrossing Connector

Column No.	a	a_1	a_2	a_3	b	b_1	b_2	b_3
# 2	2.0	-1.86	-3.29	-0.440	2.0	-1.86	-3.29	-0.440
# 3,4	2.0	-1.82	-2.47	0.352	2.0	-1.82	-2.47	0.352
# 5	2.0	-2.44	-3.89	-0.455	2.0	-2.44	-3.89	-0.455
# 6	2.0	-1.16	-2.47	-0.317	2.0	-1.16	-2.47	-0.317

Table 12 Frequencies and Periods of Figueroa Street Undercrossing Connector — Zero Friction at Expansion Joint

Mode No.	Frequency (rad/sec)	Period (sec)
1	7.072	0.889
2	9.208	0.682
3	10.075	0.624
4	11.866	0.530
5	13.924	0.451
6	15.382	0.409
7	20.080	0.313
8	22.986	0.273
9	24.171	0.260
10	26.921	0.233

Table 13 Parameter Studies of Figueroa Street Undercrossing Connector

Parameters / Case No.	Ground Excitations (OVH Ground Acceleration)	Column Type (Elastic vs. Elasto-Plastic)
Case 1	Severe Shaking: 0.5g Transverse, 0.3g Vertical	Elastic
Case 2	Severe Shaking: 0.5g Transverse, 0.3g Vertical	Elasto-Plastic
Case 3	Moderate Shaking: 0.3g Transverse, 0.18g Vertical	Elasto-Plastic
Straight	Severe Shaking: 0.5g Transverse, 0.3g Vertical	Elasto-Plastic

Table 14 Frequencies and Periods of Straight Figueroa Street Undercrossing Connector - Zero Friction at Expansion Joint

Mode No.	Frequency (rad/sec)	Period (sec)
1	7.213	0.871
2	9.315	0.675
3	10.098	0.622
4	13.921	0.451
5	13.965	0.450
6	15.367	0.409
7	20.315	0.309
8	23.390	0.269
9	26.481	0.237
10	28.081	0.224

Table 15 Maximum Horizontal Accelerations (Absolute) at the Top of Columns of 5/14 South Connector Overcrossing

Column / Case No.	#2 (g)	#3 (g)	#4 (g)	#5 (g)	#6 (g)	#7 (g)
Case 1	0.93	0.86	0.62	0.87	0.83	0.64
Case 2	0.94	1.96	1.01	0.99	1.81	0.85
Case 4	0.44	0.64	0.36	0.88	0.53	0.40
Case 5	1.01	0.83	0.70	0.85	0.72	0.51

Table 16 Maximum Horizontal Displacements at the Top of Columns of 5/14 South Connector Overcrossing

Col. / Case No.	#2 (ft)	#3 (ft)	#4 (ft)	#5 (ft)	#6 (ft)	#7 (ft)
Case 1	1.21	2.50	2.57	1.96	1.91	1.03
Case 2	1.45	3.32	3.70	3.07	1.30	0.90
Case 4	0.69	1.41	1.69	1.47	0.68	0.48
Case 5	0.78	1.69	1.94	1.80	1.28	0.74

Table 17 Maximum Horizontal Accelerations (Absolute) at the Top of Columns of Figueroa Street Undercrossing Connector

Case No. \ Column	#2 (g)	#3 (g)	#4 (g)	#5 (g)	#6 (g)
Case 1	1.01	0.94	1.38	1.06	0.68
Case 2	0.75	0.74	0.85	0.76	0.58
Case 3	0.49	0.51	0.55	0.39	0.44
Straight	0.86	0.84	0.91	0.61	0.72

Table 18 Maximum Horizontal Displacements at the Top of Columns of Figueroa Street Undercrossing Connector

Case No. \ Column	#2 (ft)	#3 (ft)	#4 (ft)	#5 (ft)	#6 (ft)
Case 1	0.17	0.32	0.68	0.61	0.31
Case 2	0.13	0.23	0.56	0.40	0.23
Case 3	0.11	0.15	0.33	0.26	0.15
Straight	0.18	0.34	0.91	0.68	0.28

Table 19 Maximum Vertical Accelerations (Absolute) at the Center of Spans of 5/14 South Connector Overcrossing

Span / Case No.	#2 (g)	#3 (g)	#4 (g)	#6 (g)	#7 (g)	#8 (g)
Case 1	0.71	0.59	0.62	0.57	0.73	0.53
Case 2	0.71	0.62	0.59	0.54	0.71	0.44
Case 4	0.41	0.38	0.29	0.30	0.44	0.28
Case 5	0.21	0.15	0.16	0.25	0.16	0.12

Table 20 Maximum Vertical Displacements at the Center of Spans of 5/14 South Connector Overcrossing

Span / Case No.	#2 (ft)	#3 (ft)	#4 (ft)	#6 (ft)	#7 (ft)	#8 (ft)
Case 1	0.091	0.106	0.138	0.047	0.051	0.084
Case 2	0.071	0.118	0.135	0.096	0.073	0.085
Case 4	0.055	0.076	0.085	0.047	0.034	0.059
Case 5	0.055	0.047	0.054	0.042	0.022	0.029

Table 21 Maximum Vertical Accelerations (Absolute) at the Center of Spans of Figueroa Street Undercrossing Connector

Span / Case No.	#2 (g)	#3 (g)	#4 (g)	#5 (g)	#6 (g)
Case 1	0.90	0.77	1.60	1.70	0.95
Case 2	1.00	0.58	1.65	1.65	0.86
Case 3	0.55	0.39	0.97	0.99	0.56
Straight	1.12	0.92	1.30	2.64	0.99

Table 22 Maximum Vertical Displacement at the Center of Spans of Figueroa Street Undercrossing Connector

Span / Case No.	#2 (ft)	#3 (ft)	#4 (ft)	#5 (ft)	#6 (ft)
Case 1	0.046	0.129	0.117	0.262	0.253
Case 2	0.051	0.123	0.161	0.254	0.238
Case 3	0.026	0.077	0.093	0.147	0.152
Straight	0.054	0.155	0.153	0.375	0.241

Table 23 Flexural Yield Rotations of Columns of 5/14 South Connector Overcrossing

Column No.	E (k/ft^2)	I_y (ft^4)	M_{yo} 10^4(k-ft)	h_y (ft)	Θ_y^Y 10^{-3}(rad)	I_z (ft^4)	M_{zo} 10^4(k-ft)	h_z (ft)	Θ_z^Y 10^{-3}(rad)
# 2	490,000	86.0	1386	5.0	1.65	327.6	2537	10.0	1.59
# 3,5	490,000	143.2	1671	6.0	1.43	379.6	2598	10.0	1.40
# 4	524,000	143.2	2156	6.0	1.48	379.6	3331	10.0	1.69
# 6	524,000	49.6	1102	4.0	1.60	275.3	2517	10.0	1.75
# 7,8	557,000	49.6	1320	4.0	1.69	275.3	2997	10.0	1.96

Table 24 Flexural Yield Rotations at the Base of Columns of Figueroa Street Undercrossing Connector

Column No.	E (k/ft^2)	I_y (ft^4)	M_{yo} 10^4 (k-ft)	h_y (ft)	Θ_y^Y 10^{-3} (rad)	I_z (ft^4)	M_{zo} 10^4 (k-ft)	h_z (ft)	Θ_z^Y 10^{-3} (rad)
# 2	432,000	163.7	2470	7.5	2.17	163.7	2470	7.5	2.17
# 3,4	432,000	163.7	2250	7.5	1.99	163.7	2250	7.5	1.99
# 5	432,000	163.7	1980	7.5	1.75	163.7	1980	7.5	1.75
# 6	432,000	163.7	3560	7.5	3.24	163.7	3560	7.5	3.24

Table 25 Maximum Local Bending Ductility Factors at the Base of Columns of 5/14 South Connector Overcrossing

Base of / Case No.	Column # 2	Column # 3	Column # 4	Column # 5	Column # 6	Column # 7
Case 1	1.3	2.7	1.3	3.2	4.6	3.6
Case 2	4.8	4.0	3.0	7.4	12.2	2.2
Case 4	1.5	1.2	1.0	3.3	2.5	1.8
Case 5	1.6	1.8	1.1	3.3	7.8	2.3

Table 26 Maximum Local Bending Ductility Factors at the Base of Columns of Figueroa Street Undercrossing Connector

Base of / Case No.	Column # 2	Column # 3	Column # 4	Column # 5	Column # 6
Case 1	-*	-	-	-	-
Case 2	1.3	4.0	5.4	4.7	-
Case 3	-	2.4	4.5	3.1	-
Straight	2.3	4.2	22.2	18.0	1.2

* No yielding occurs.

Table 27 Maximum Joint Separations and Longitudinal Restrainer Tie Ductility Factors at the Expansion Joints of 5/14 South Connector Overcrossing

Case No.	Max. Separation (ft)				Tie Ductility Factor			
	EJ #1	EJ #2	EJ #3	EJ #4	EJ #1	EJ #2	EJ #3	EJ #4
Case 1	1.21	0.92	0.40	0.21	78.8	58.4	26.2	13.4
Case 2	1.40	1.63	0.37	0.77	278.0	339.0	94.5	145.1
Case 4	0.79	0.48	0.07	0.34	154.0	101.0	28.3	63.6
Case 5]	0.94	0.63	0.40	0.22	60.4	41.2	25.4	14.4

Table 28 Maximum Joint Separations and Longitudinal Restrainer Tie Ductility Factors at the Expansion Joint of Figueroa Street Undercrossing Connector

Case No.	Joint Separation (ft)	Tie Ductility Factor
Case 1	0.305	-*
Case 2	0.237	-
Case 3	0.133	-
Straight	0.001	-

* No yielding occurs.

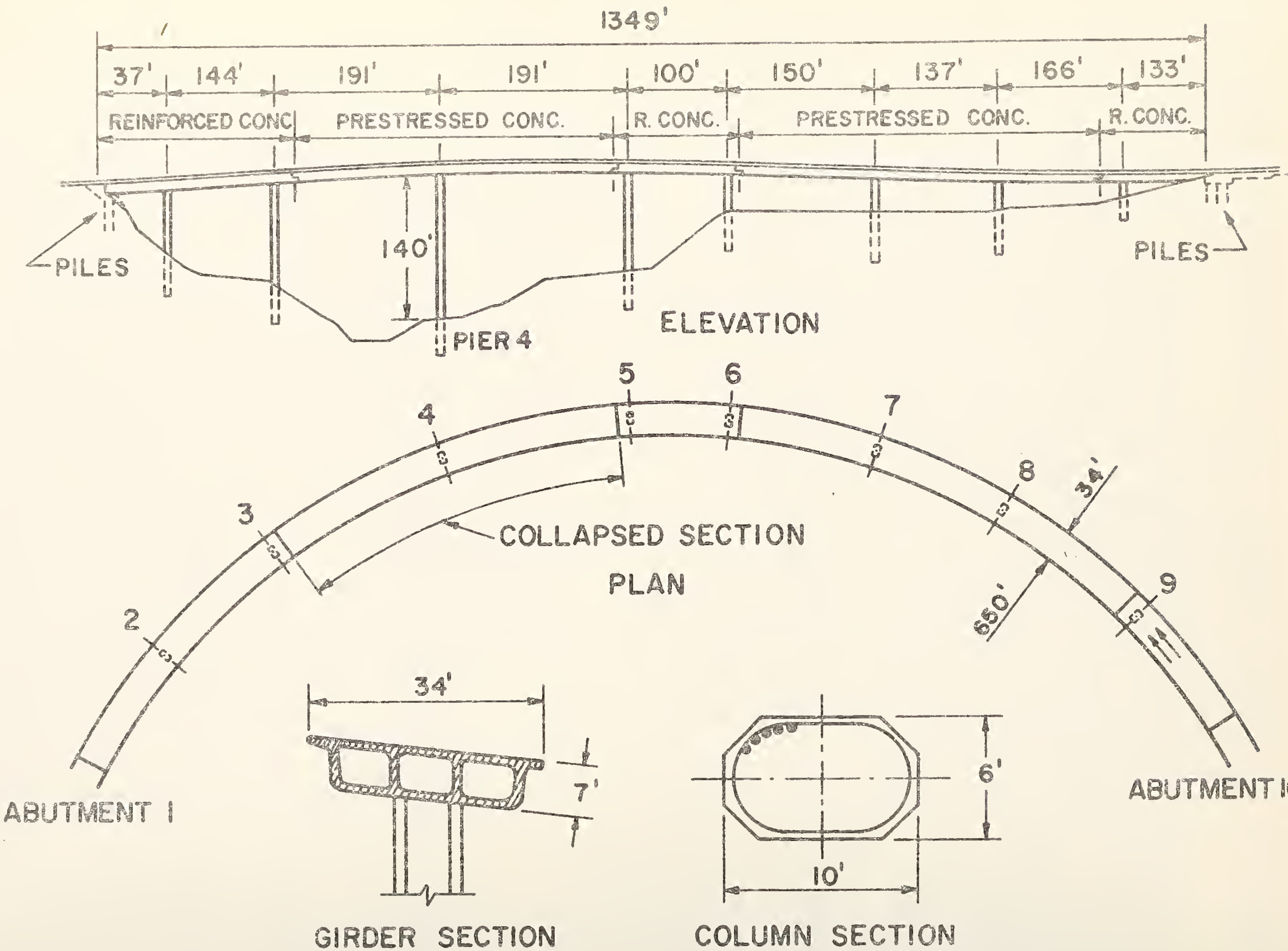


Fig. 14 The Structural System of 5/14 South Connector Overcrossing

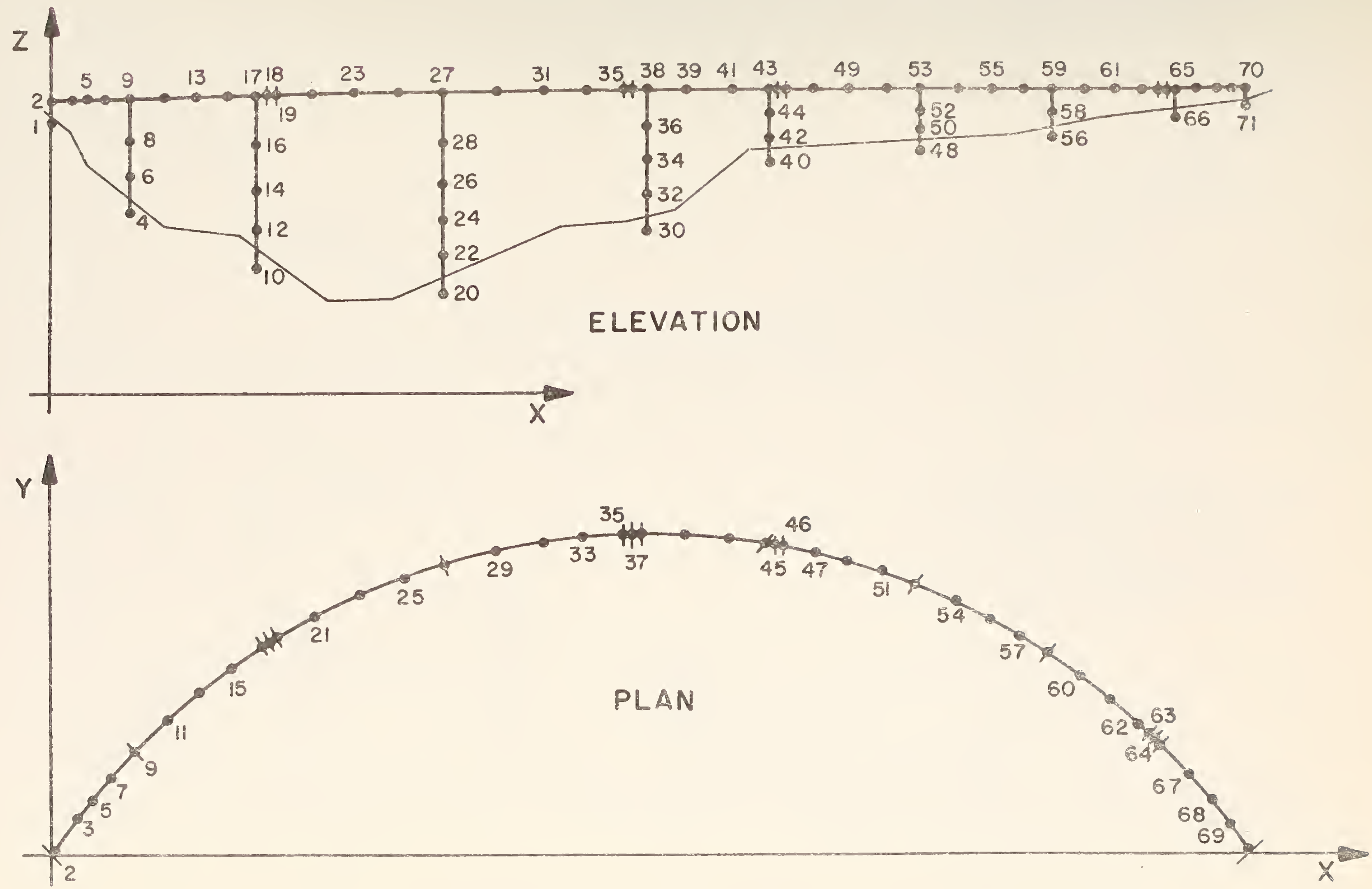


Fig. 15 Lumped Parameter System For 5/14 South Connector Overcrossing

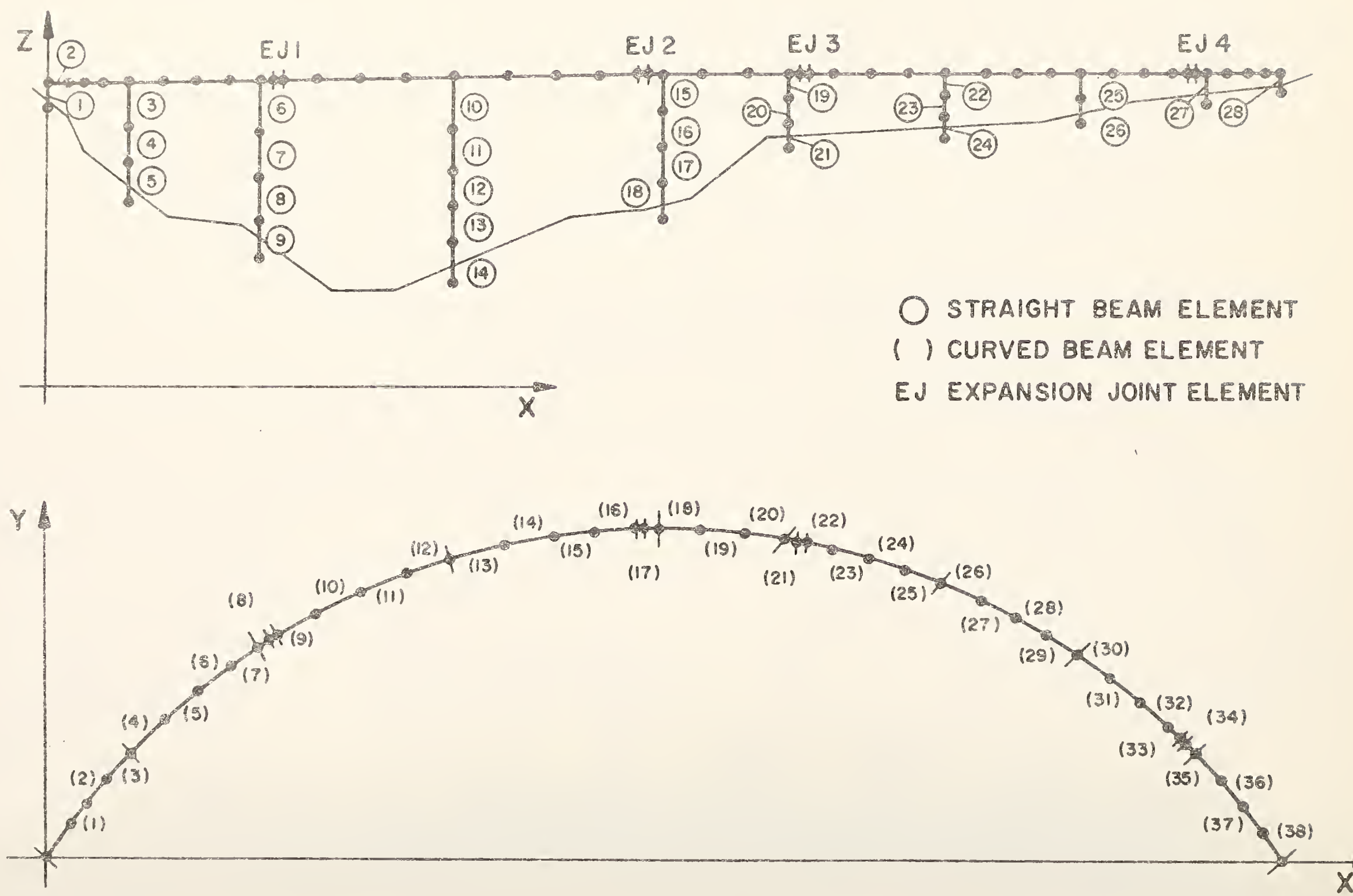


Fig. 16 Finite Element Model of 5/14 South Connector Overcrossing

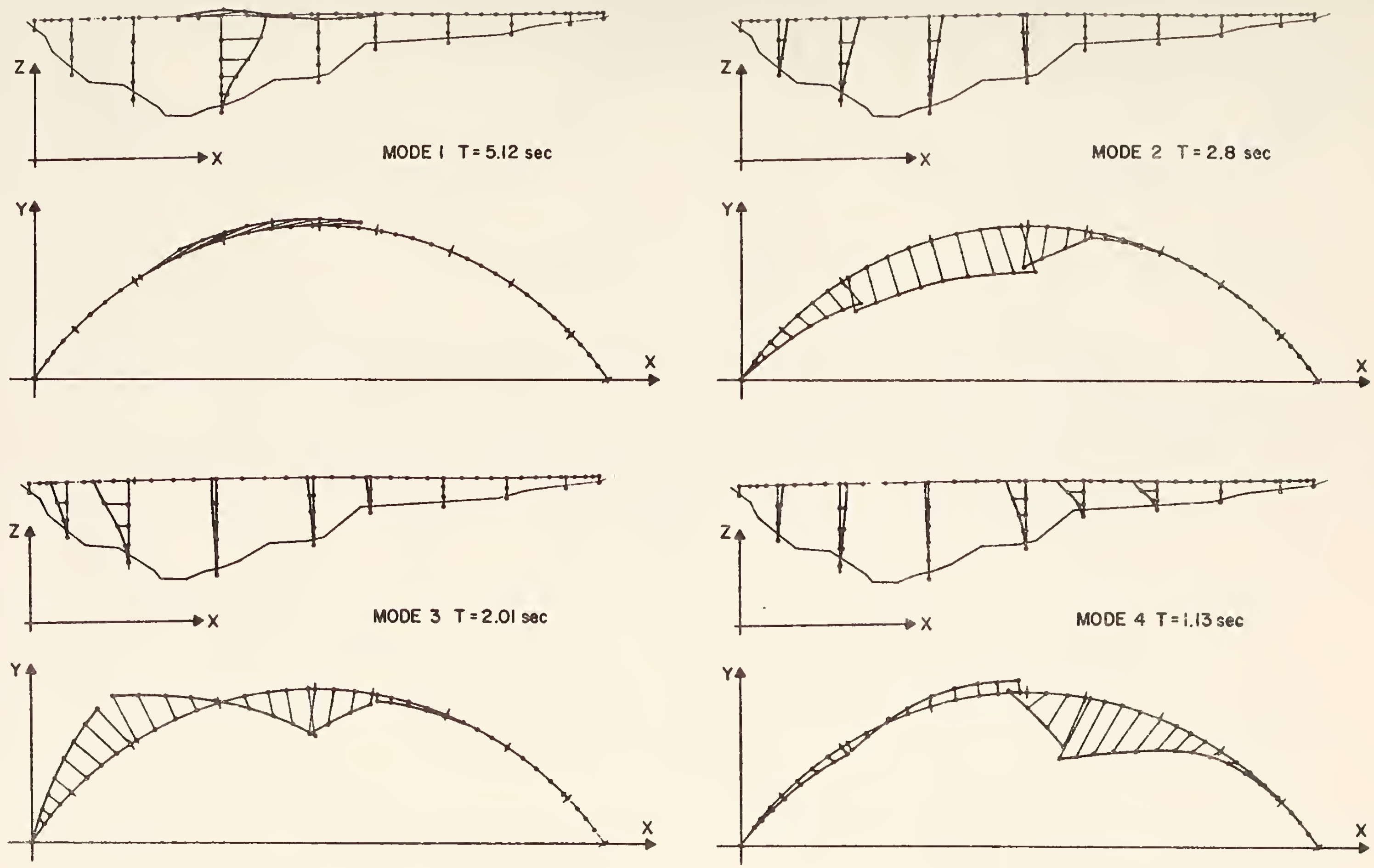


Fig. 17 Mode Shapes of 5/14 South Connector Overcrossing: Zero Friction at Expansion Joints

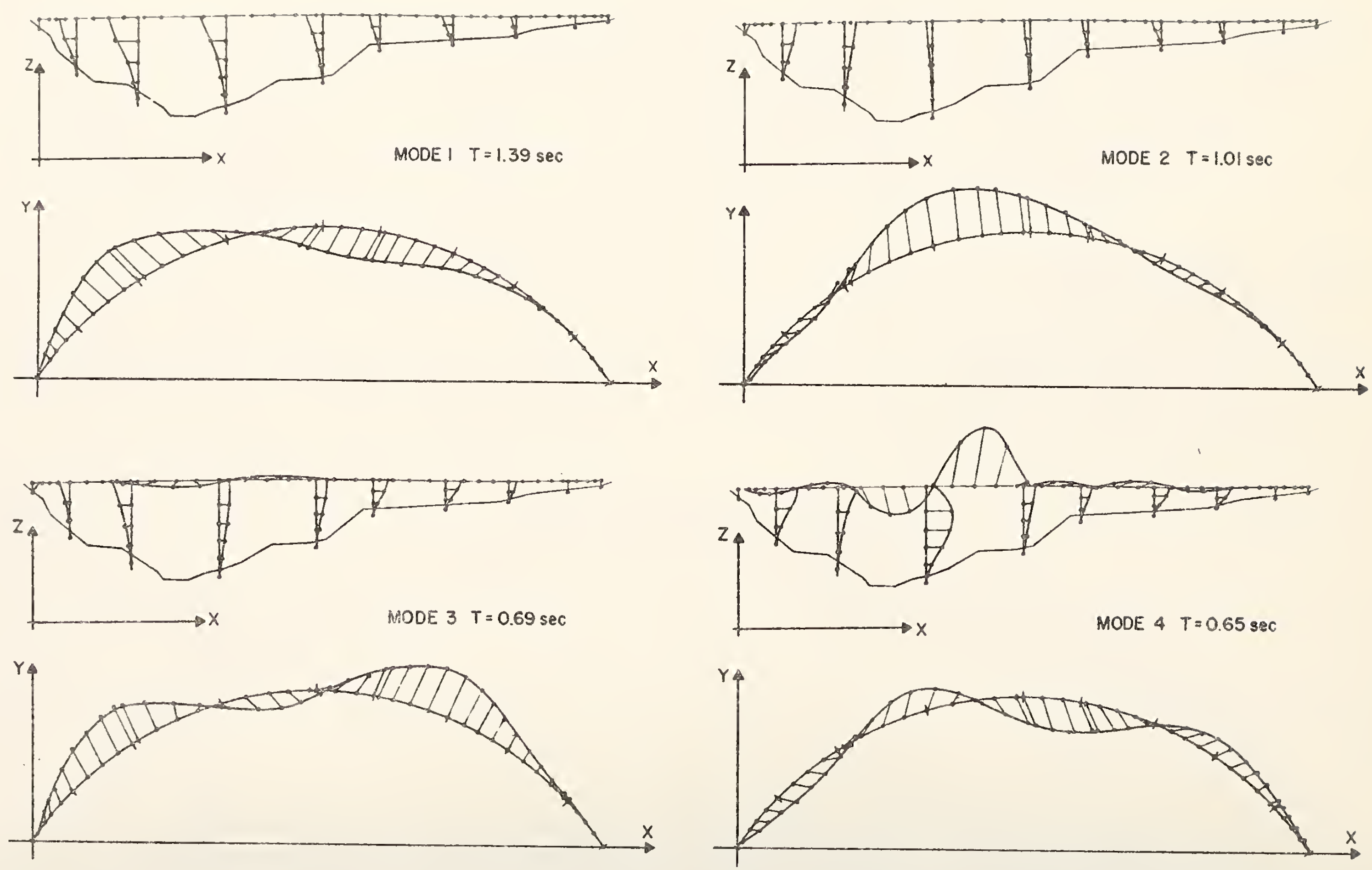


Fig. 18 Mode Shapes of 5/14 South Connector Overcrossing: Infinite Friction at Expansion Joints

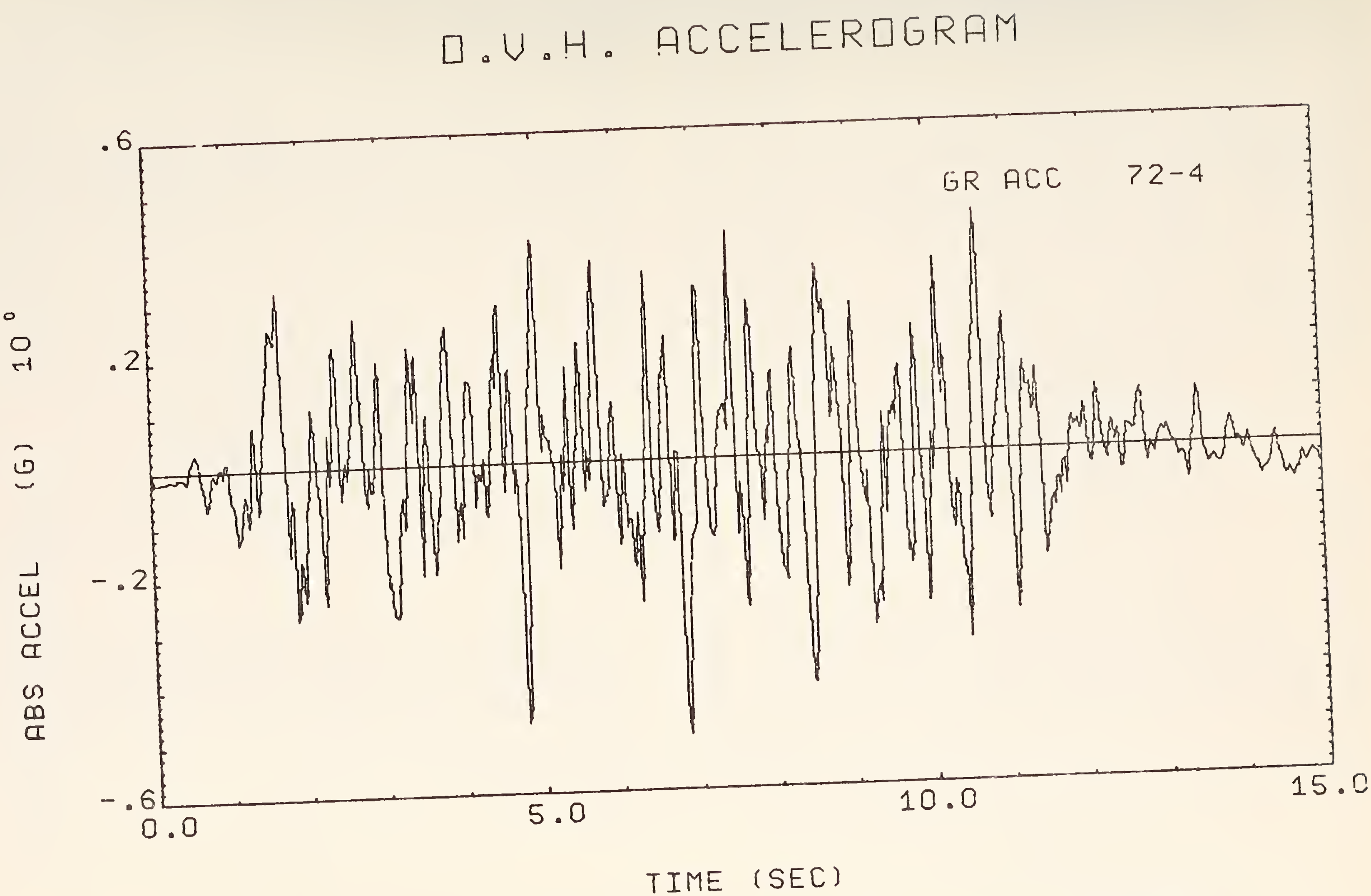


Fig. 19 Simulated Ground Acceleration Record of the San Fernando Earthquake at the Olive View Hospital Site

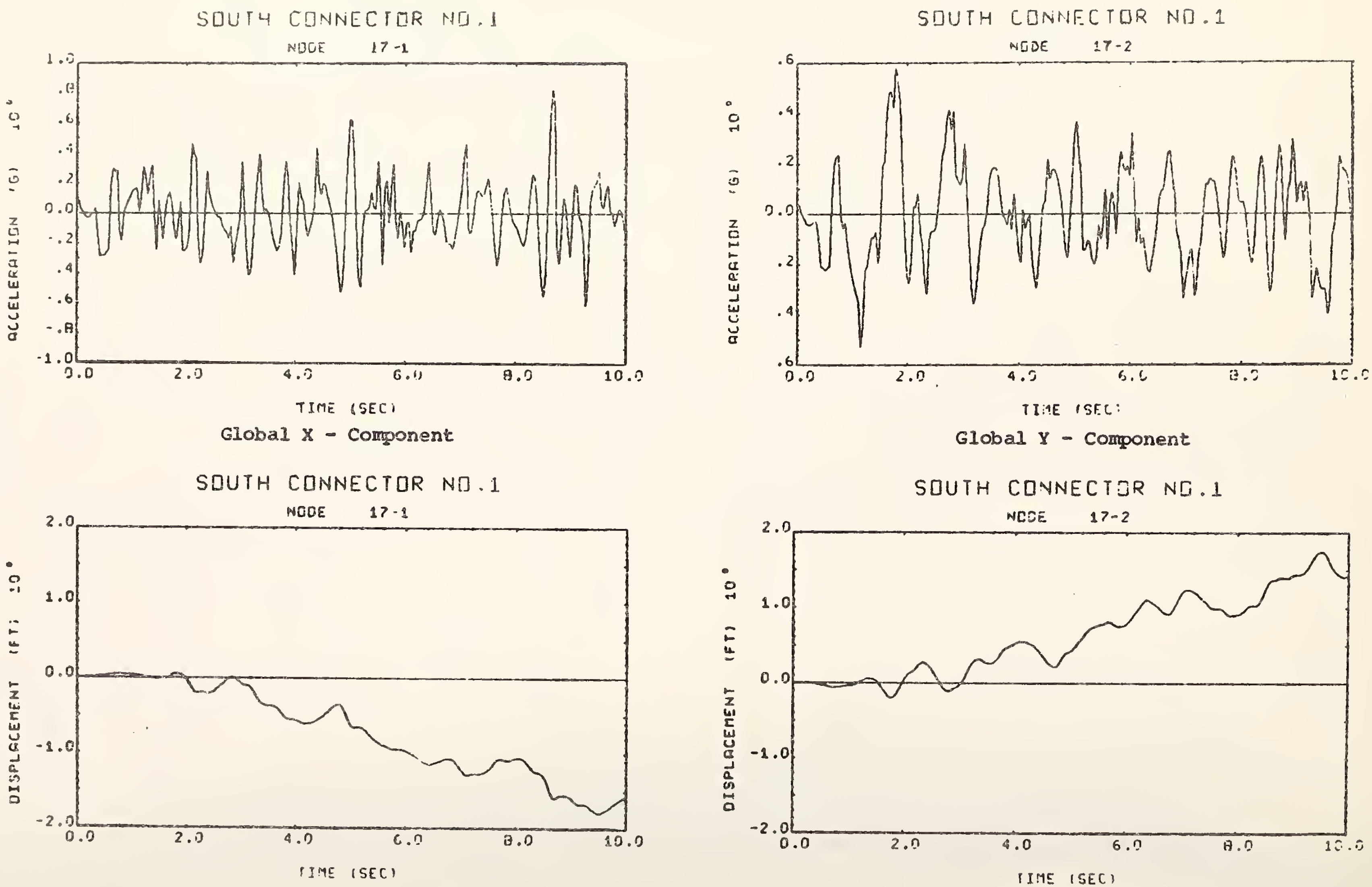
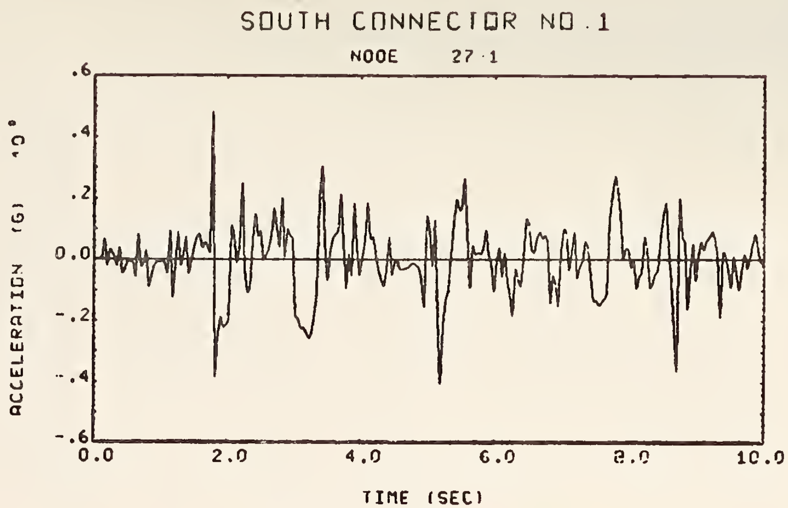
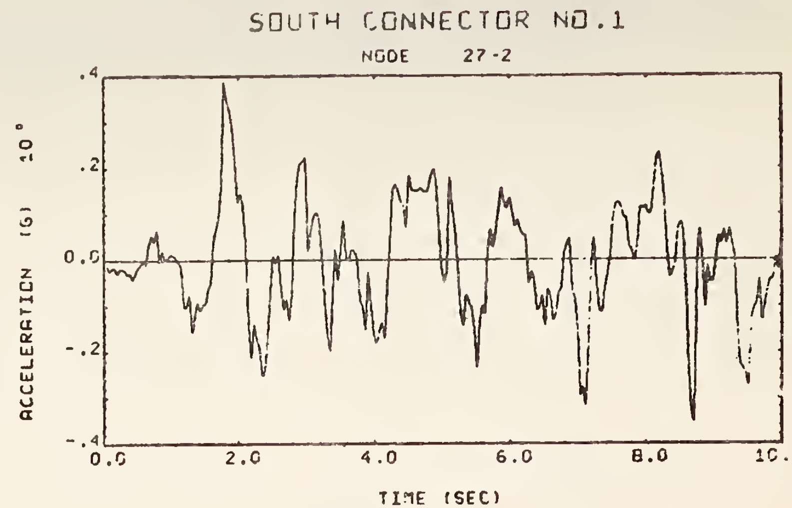


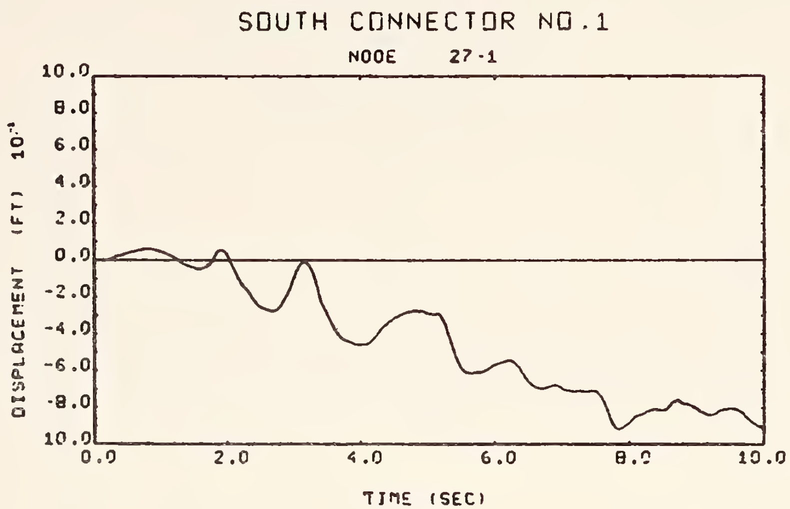
Fig. 20 Horizontal Acceleration and Displacements at the Top of Column # 3





Global X - Component

Global Y - Component



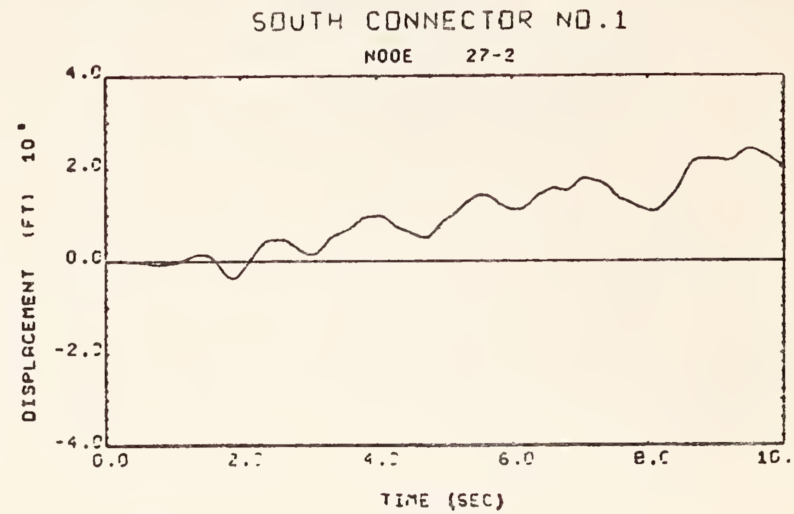


Fig. 21 Horizontal Acceleration and Displacements at the Top of Column # 4

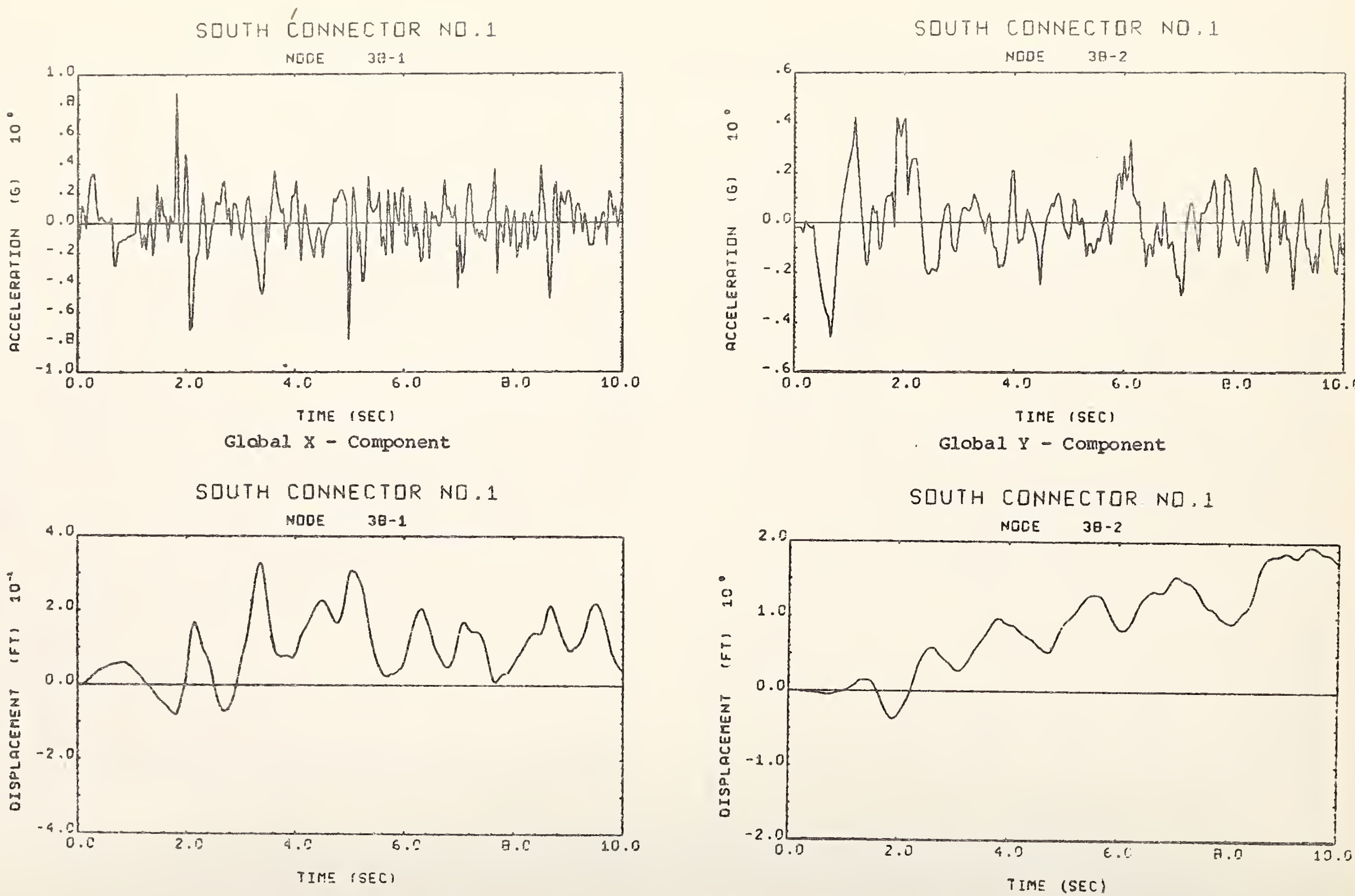
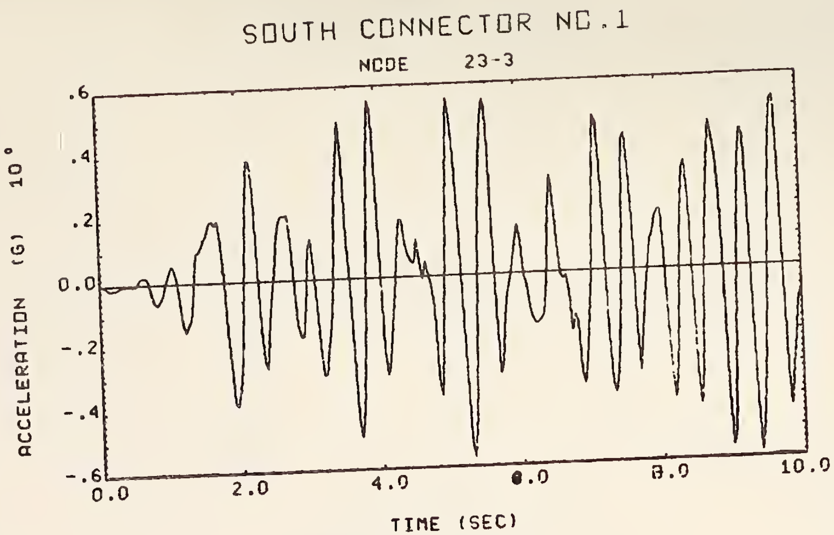
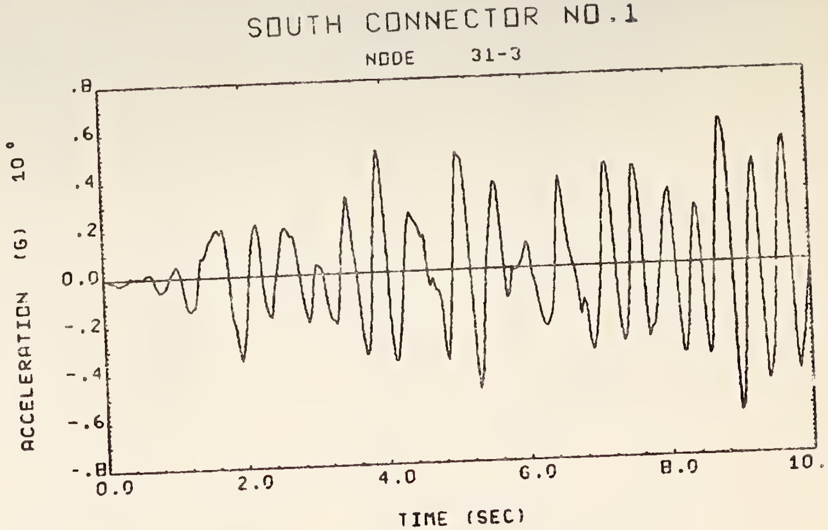


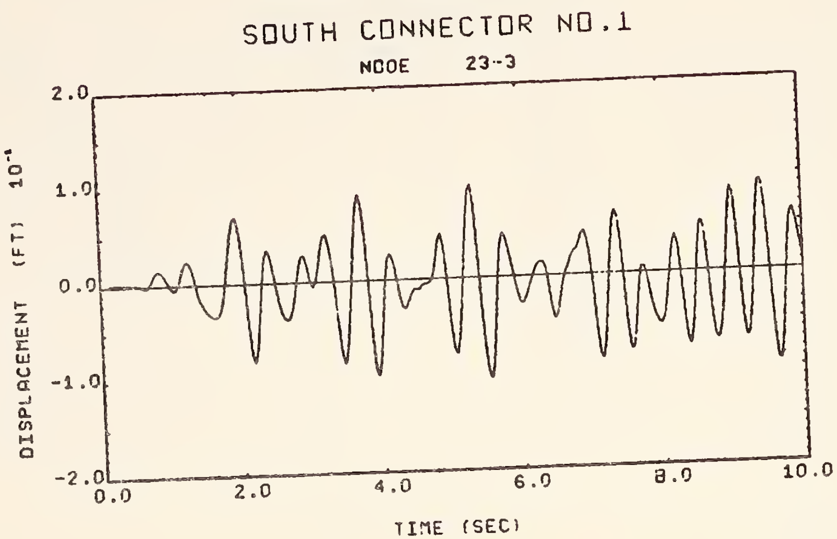
Fig. 22 Horizontal Accelerations and Displacements at the Top of Column # 5



Center of Span # 3



Center of Span # 4



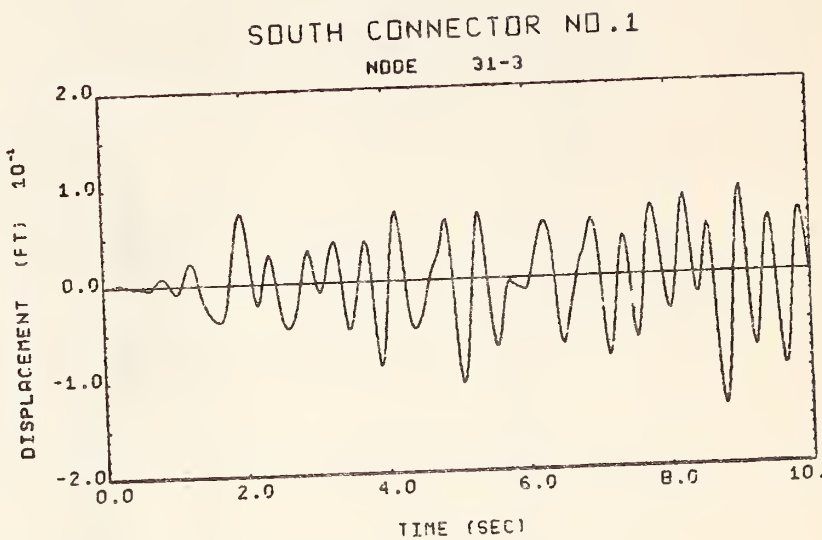
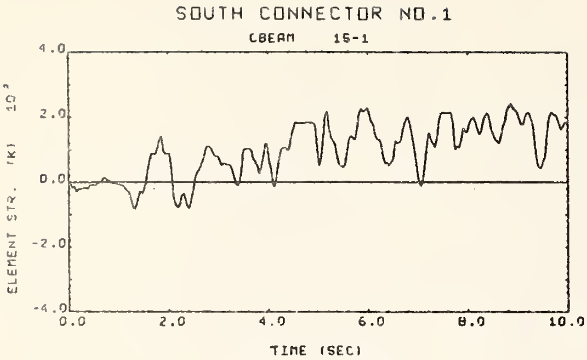
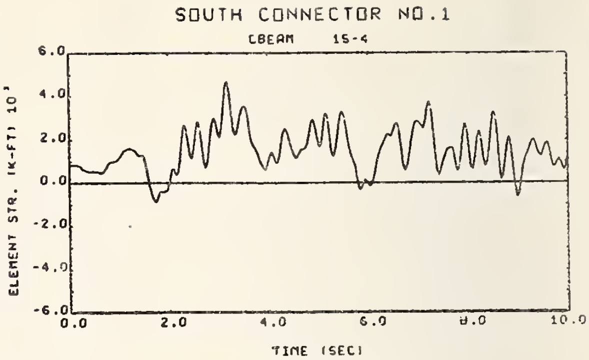


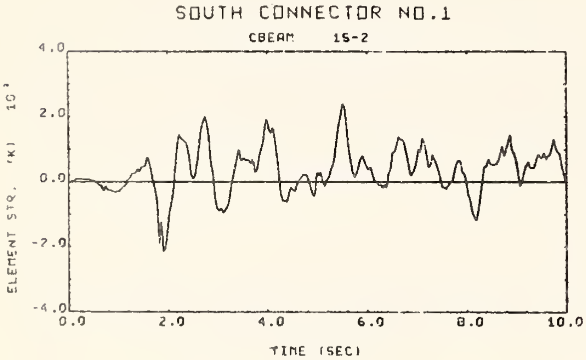
Fig. 23 Vertical Acceleration and Displacements at the Center of Spans # 3 and # 4



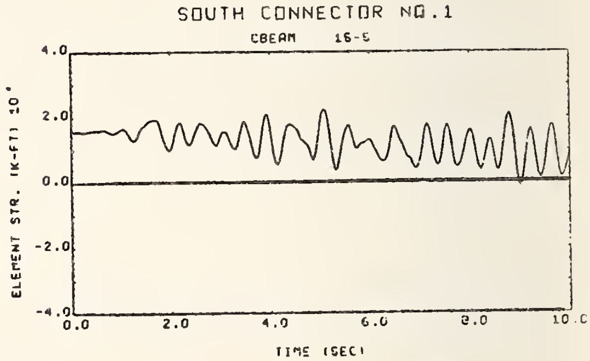
Axial Force



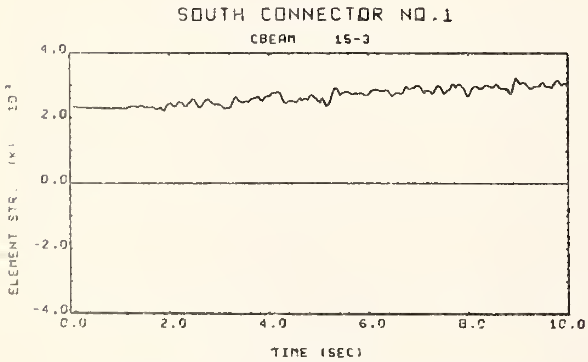
Torsional Moment



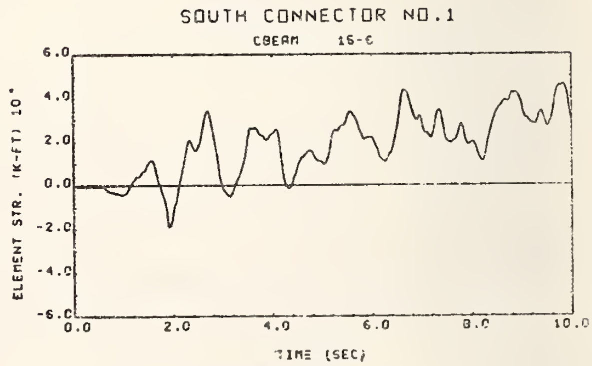
Shear in the Local y-Direction



Bending Moment about the Local y Axis



Shear in the Local z-Direction



Bending Moment about the Local z Axis

Fig. 24 Generalized Forces in the Girder at the Center of Span # 4

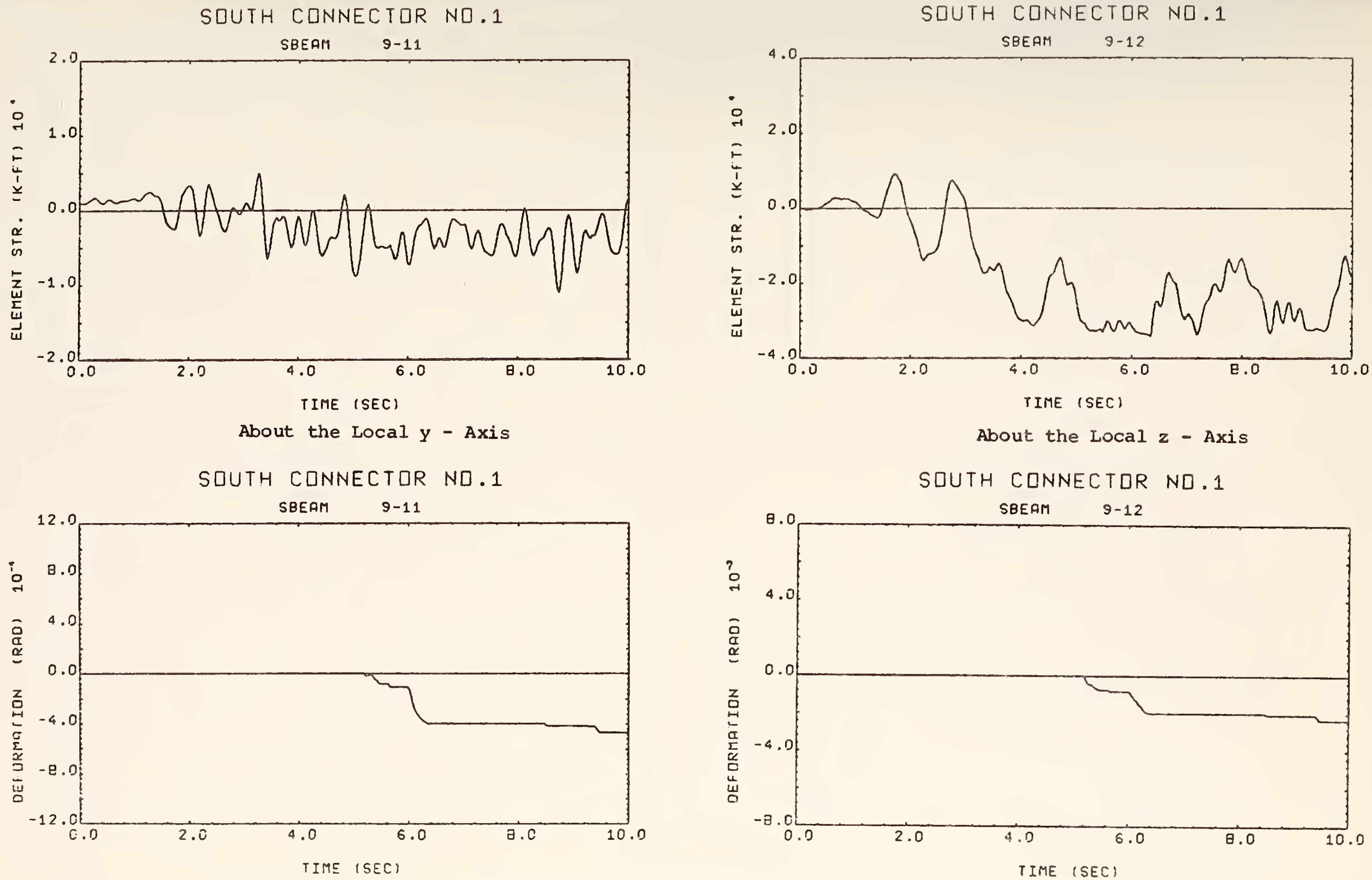


Fig. 25 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3

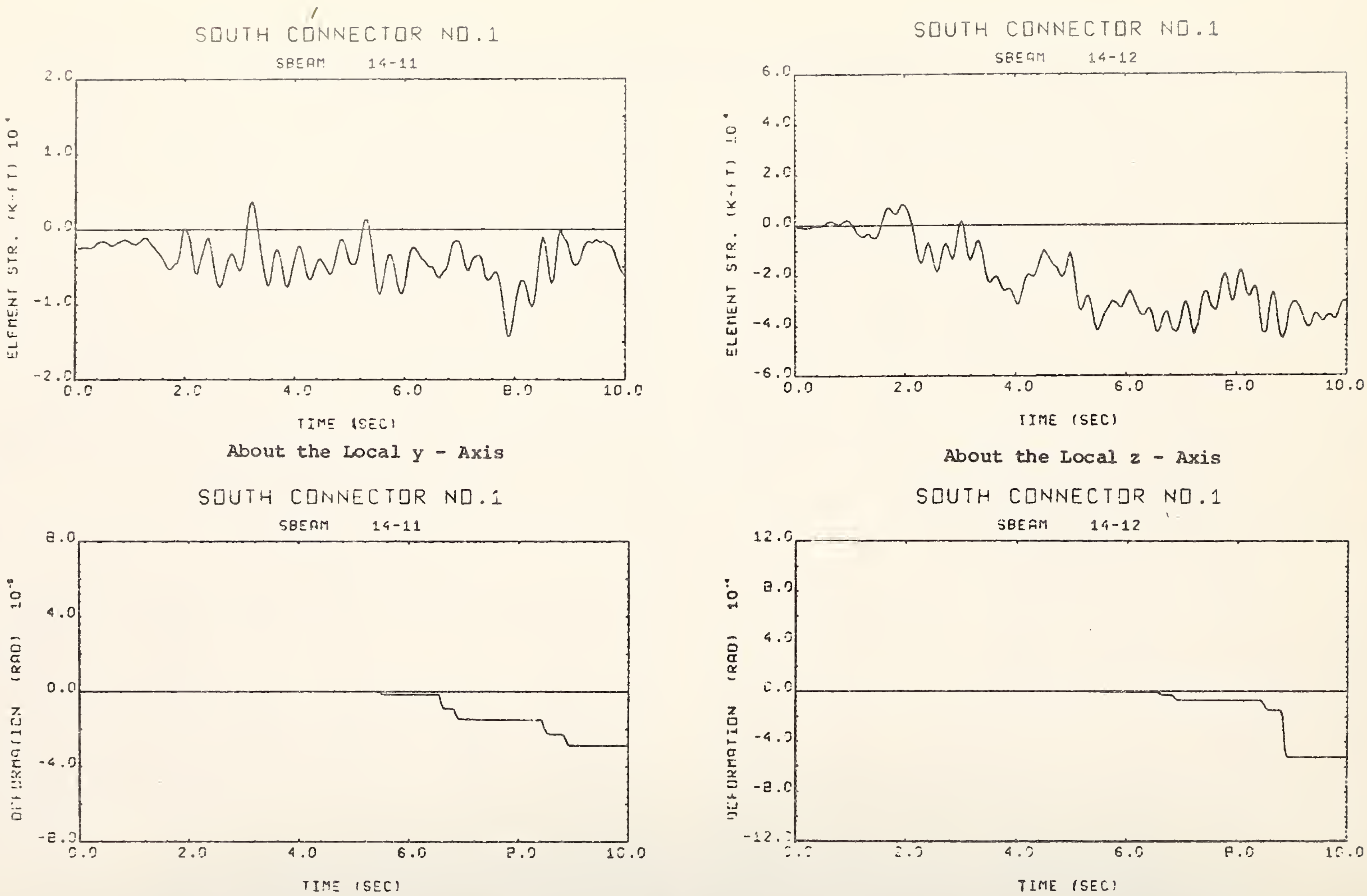


Fig. 26 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

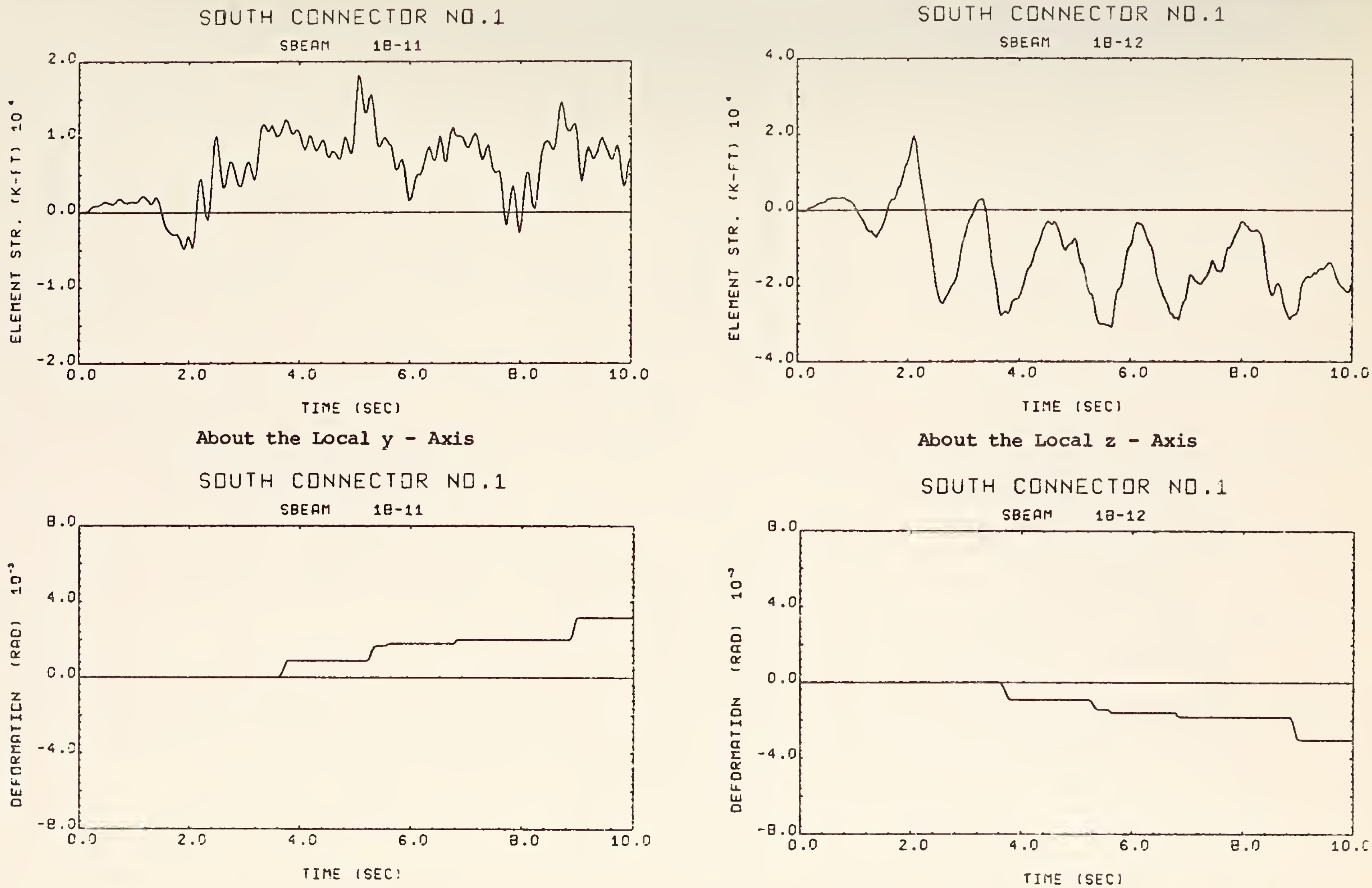


Fig. 27 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 5

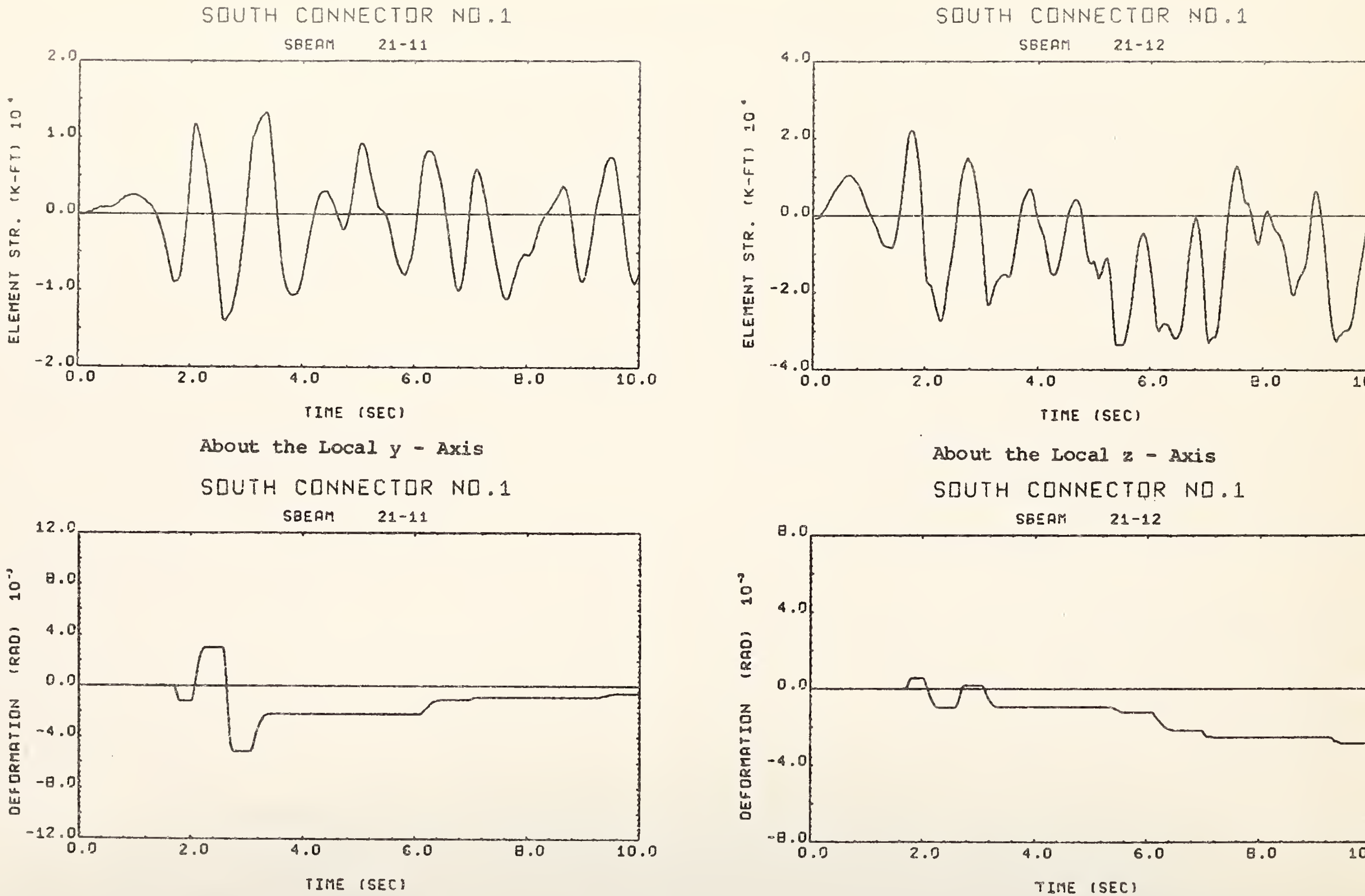
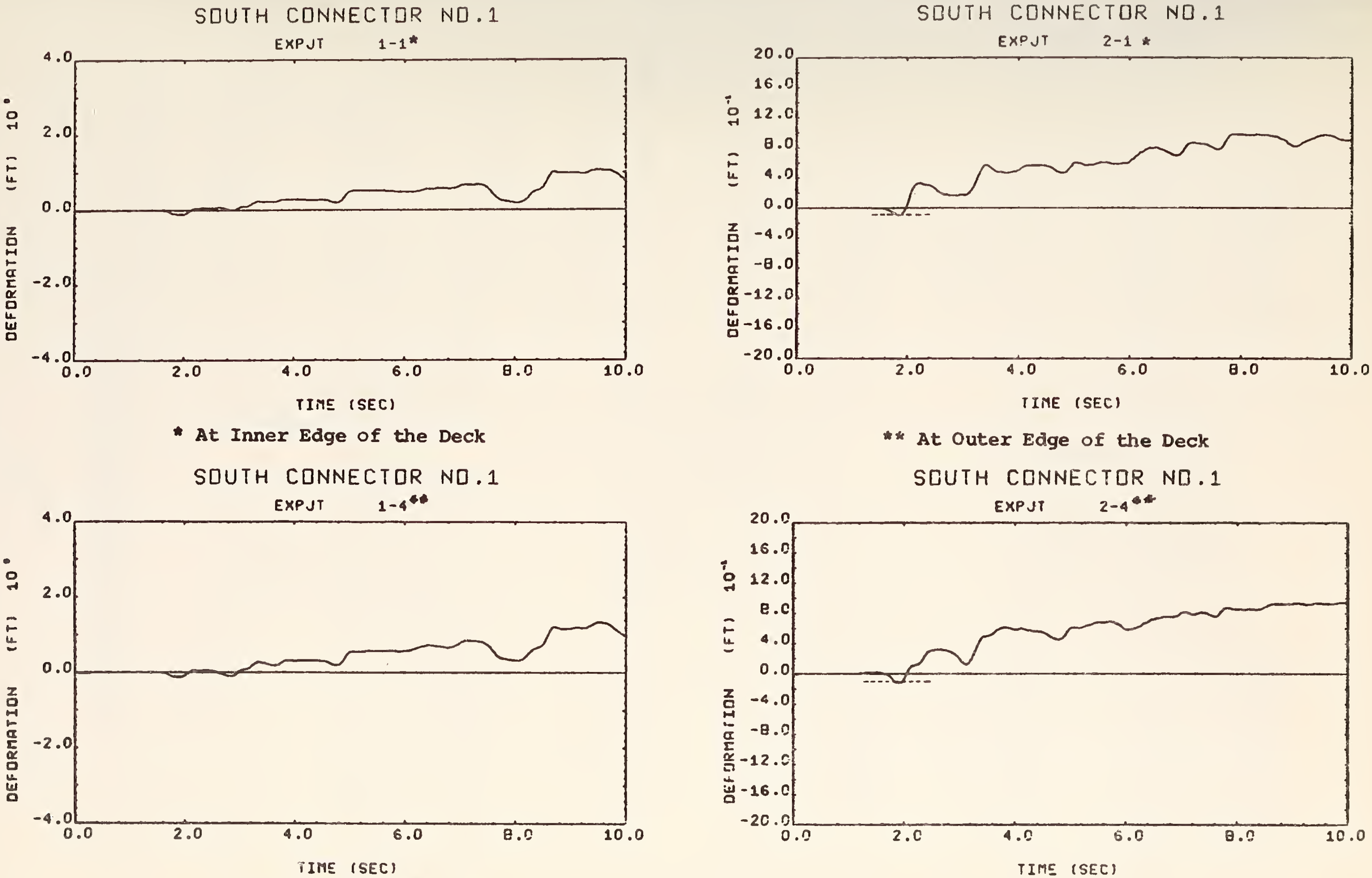


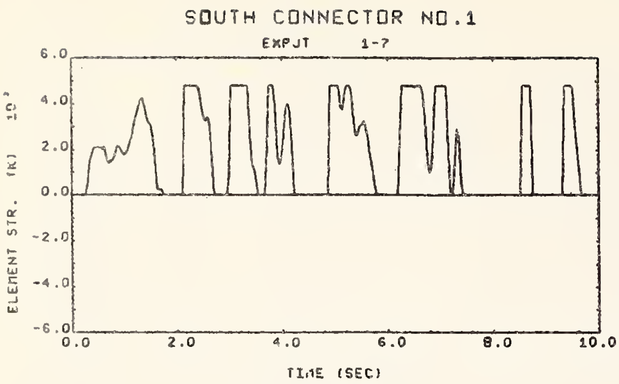
Fig. 28 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 6



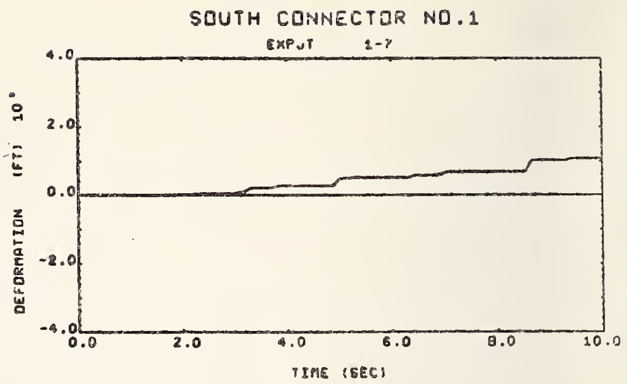
Expansion Joint # 1

Expansion Joint # 2

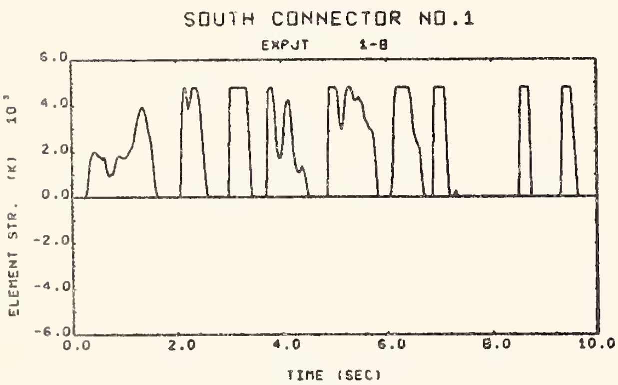
Fig. 29 Longitudinal Joint Separations at Expansion Joints # 1 and # 2



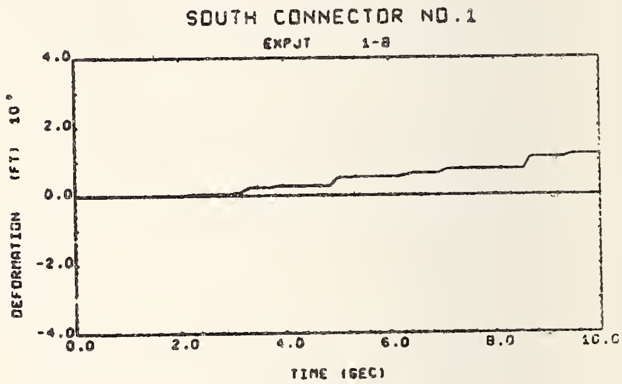
Tie Bar # 1



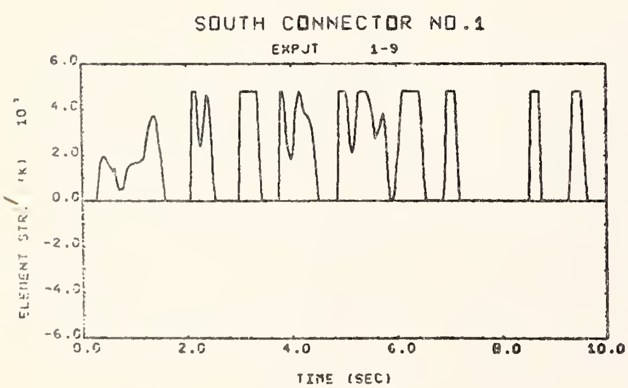
Tie Bar # 1



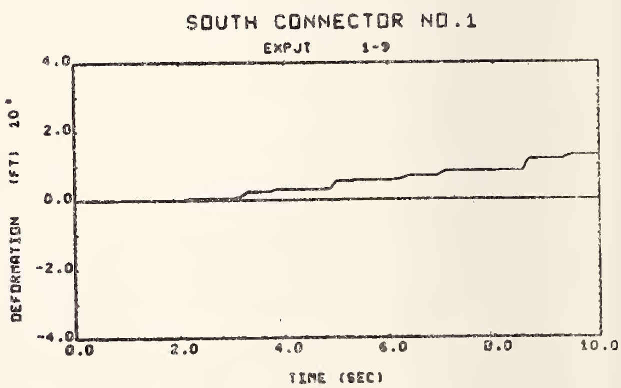
Tie Bar # 2



Tie Bar # 2



Tie Bar # 3



Tie Bar # 3

Fig. 30 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 1

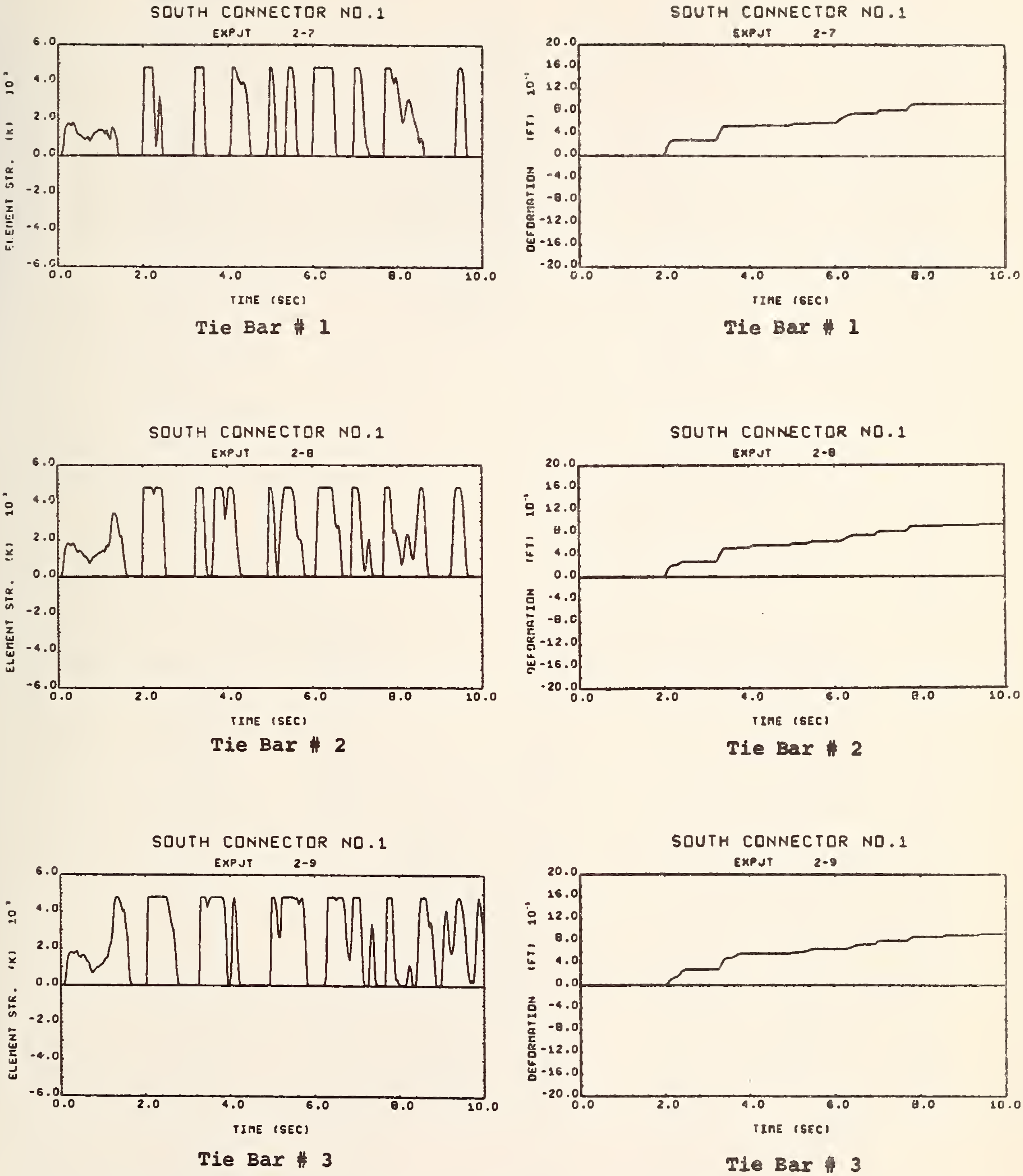


Fig. 31 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 2

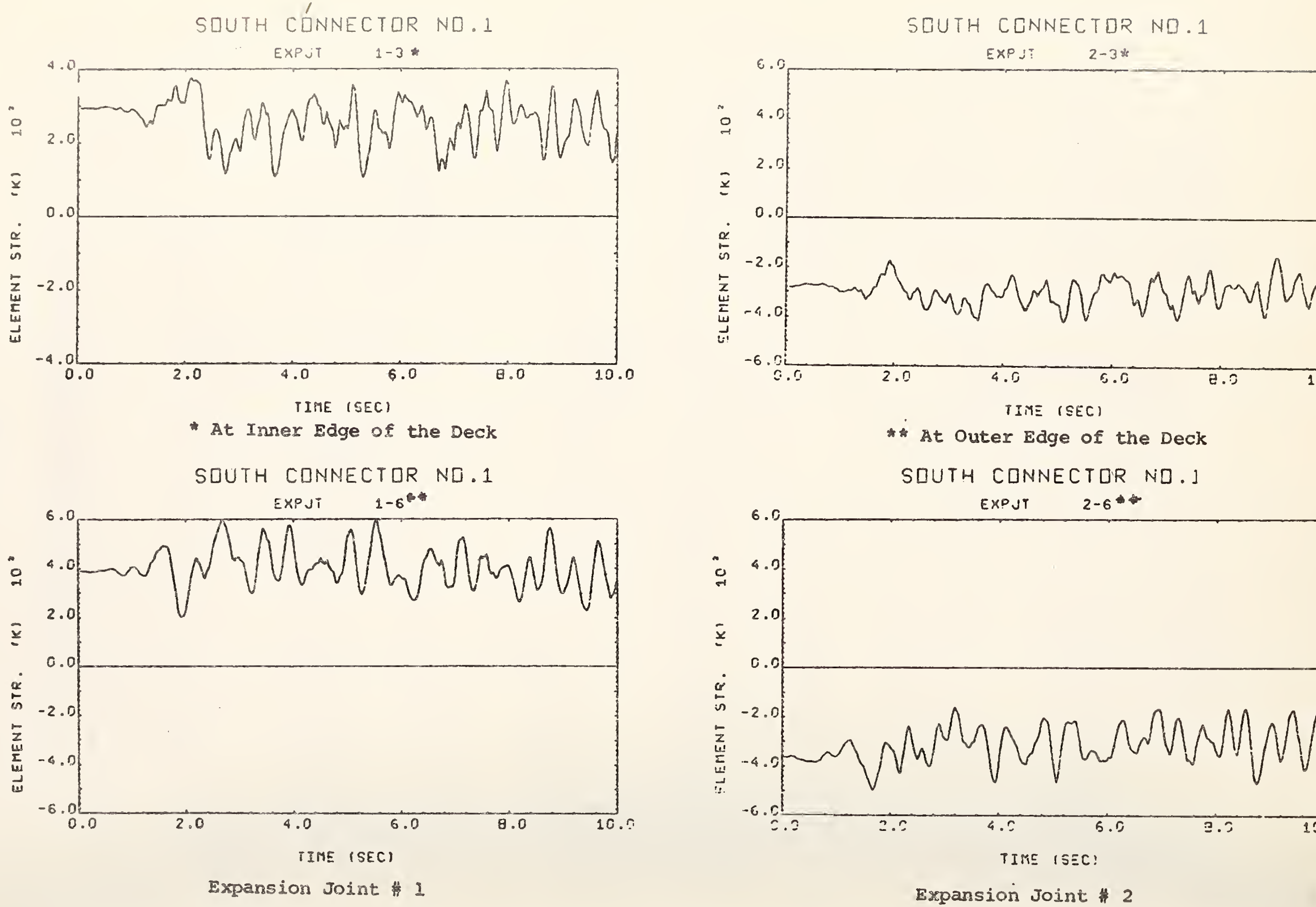


Fig. 32 Forces in the Vertical Restrainers at Expansion Joints # 1 and # 2

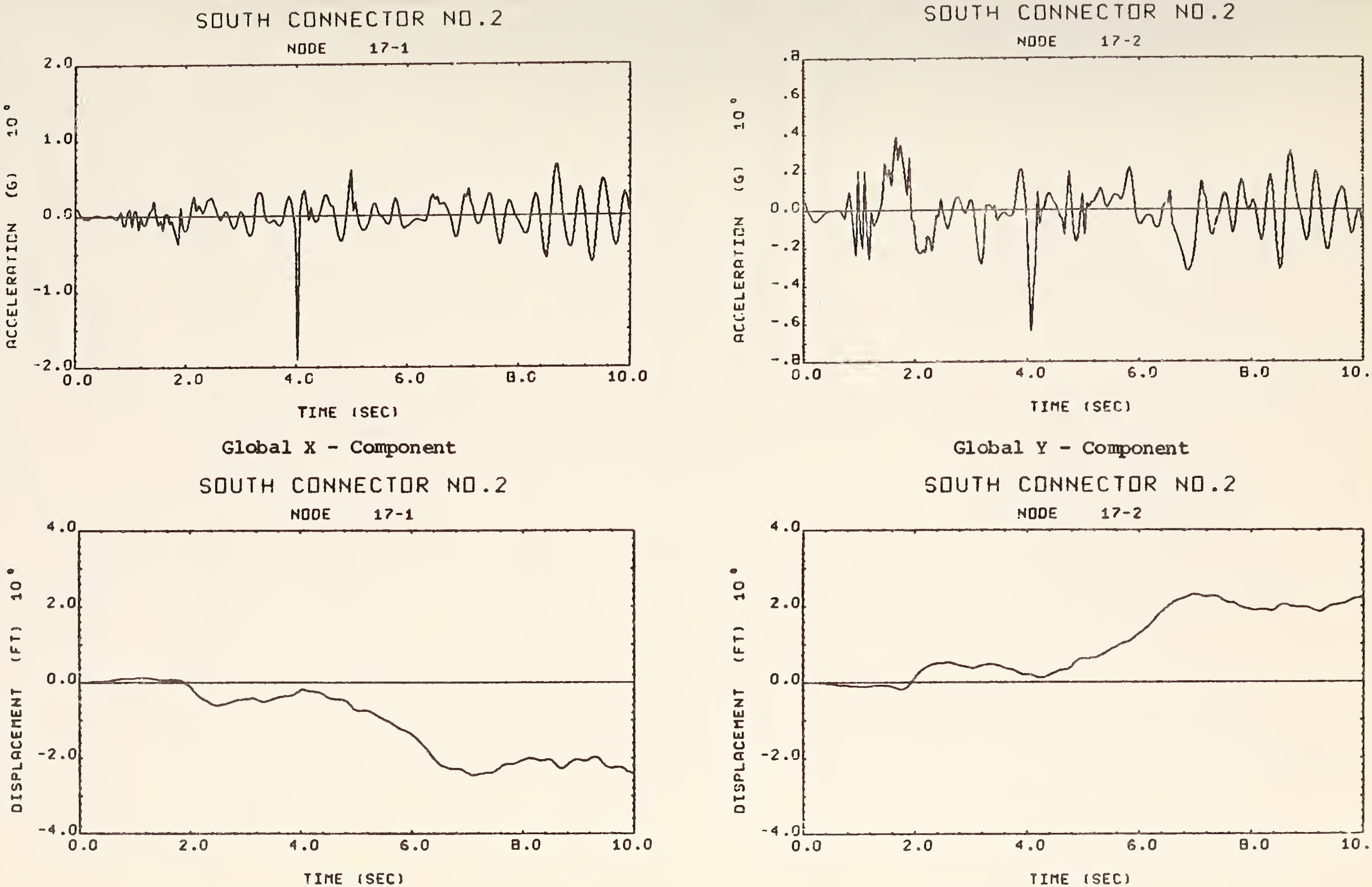
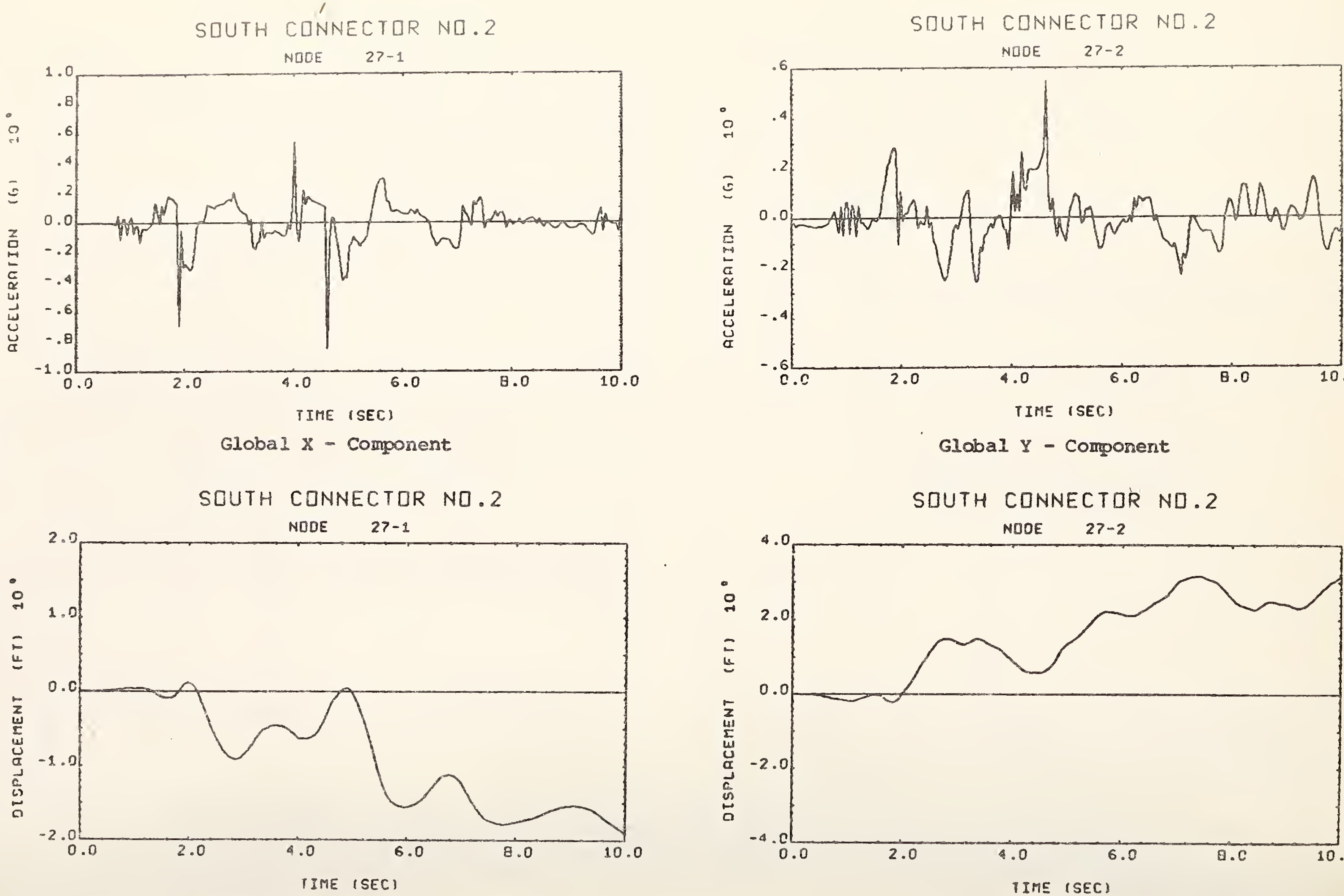


Fig. 33 Horizontal Accelerations and Displacements at the Top of Column # 3



Global X - Component

Global Y - Component

Fig. 34 Horizontal Accelerations and Displacements at the Top of Column # 4

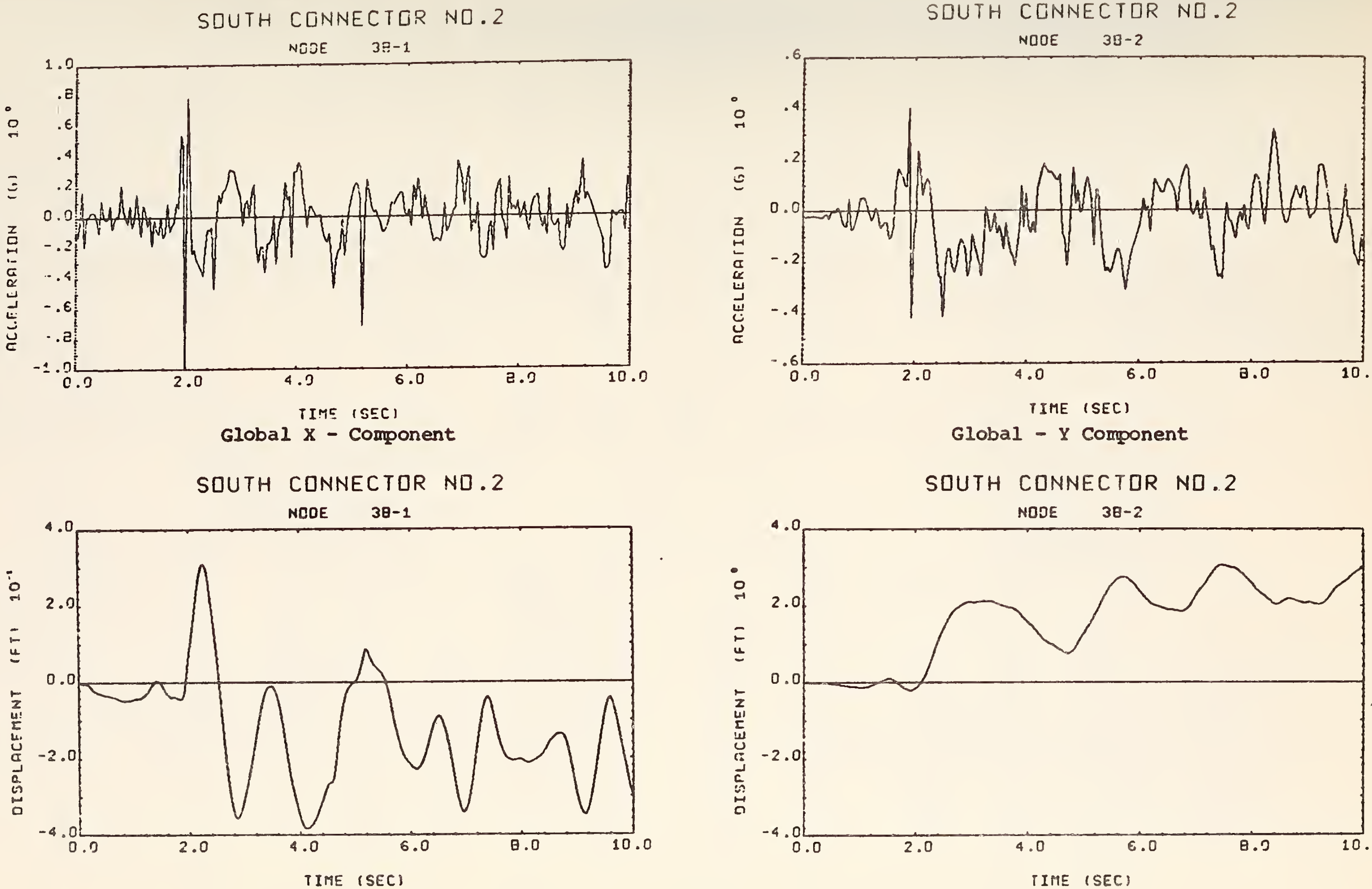


Fig. 35 Horizontal Accelerations and Displacements at the Top of Column # 5

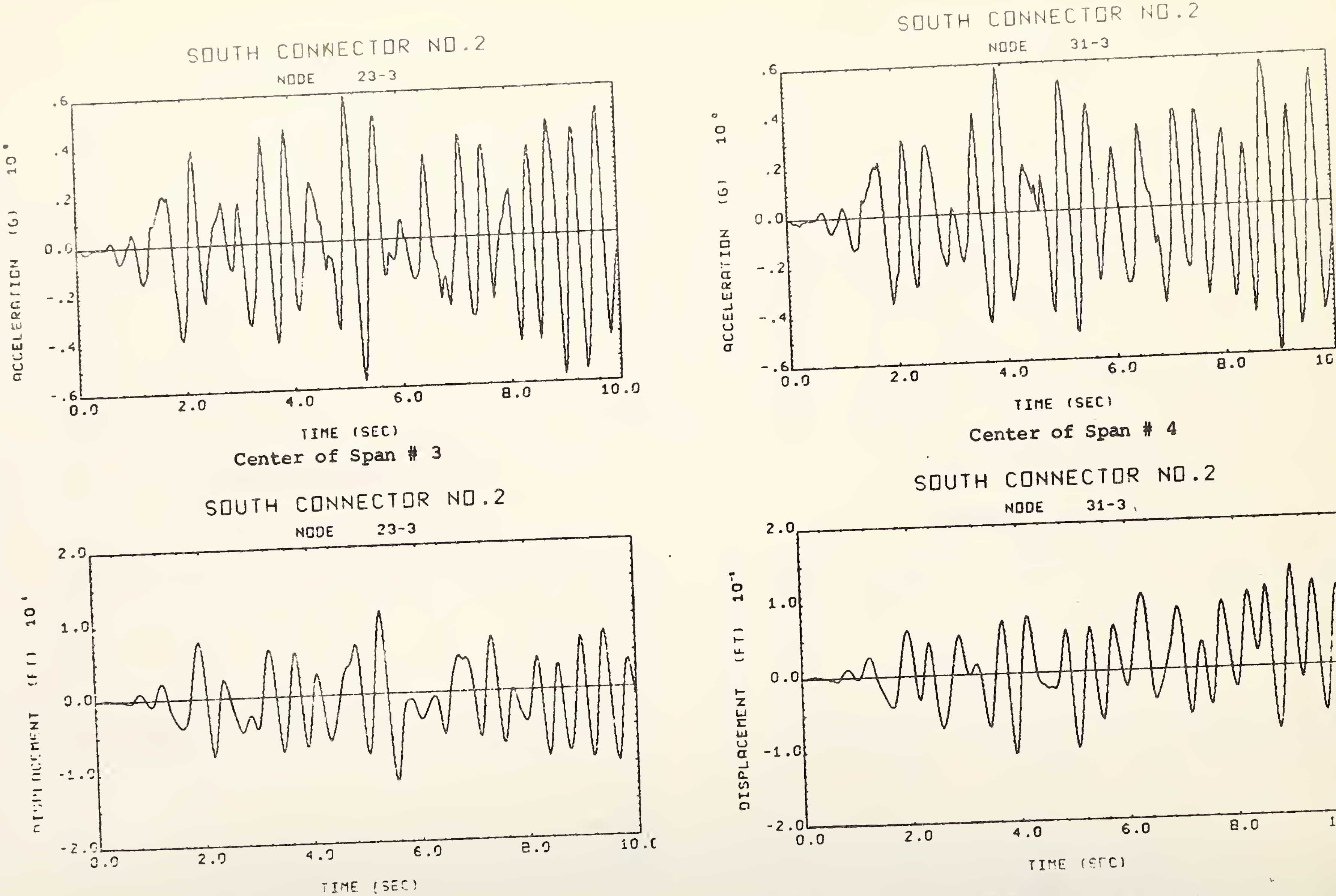
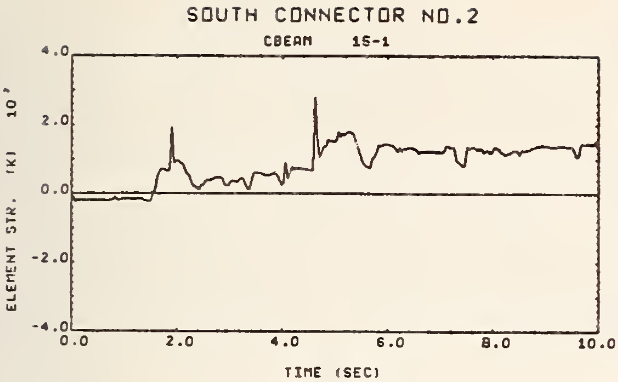
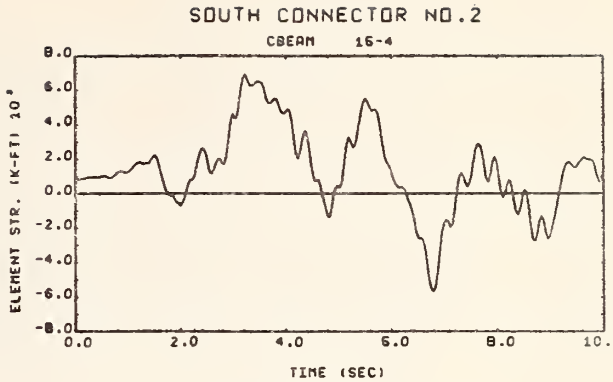


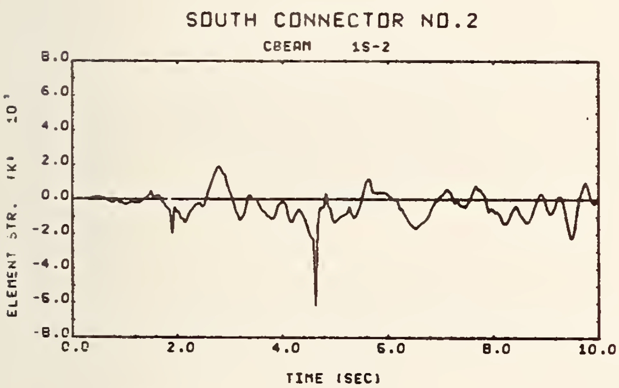
Fig. 36 Vertical Accelerations and Displacements at the Center of Spans # 3 and # 4



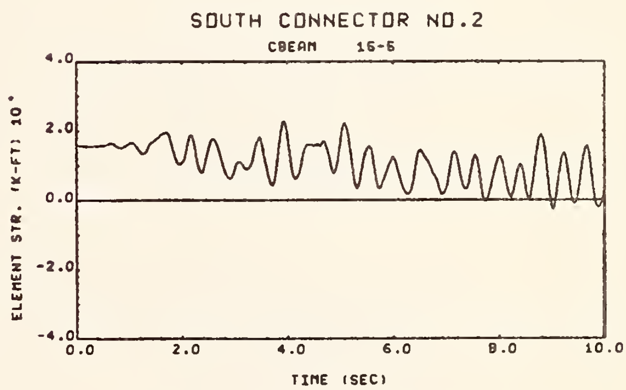
Axial Force



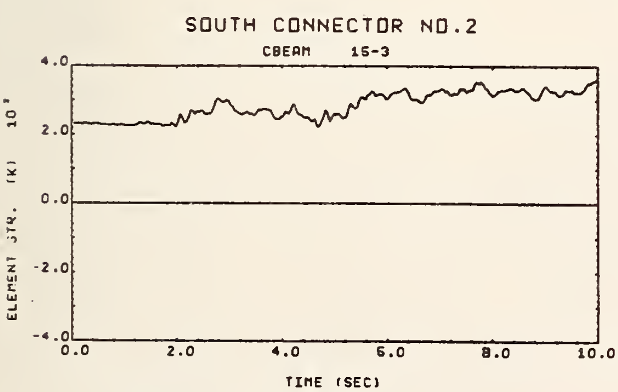
Torsional Moment



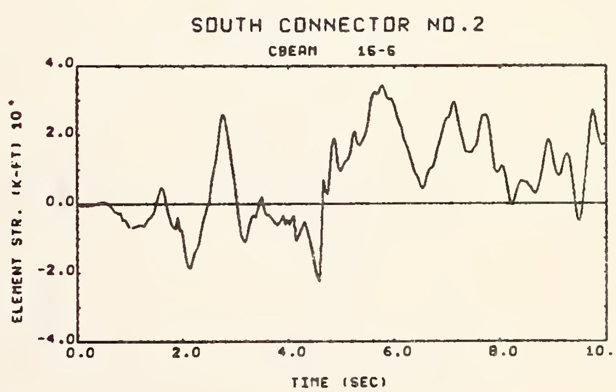
Shear in the Local y - Direction



Moment about the Local y - Axis

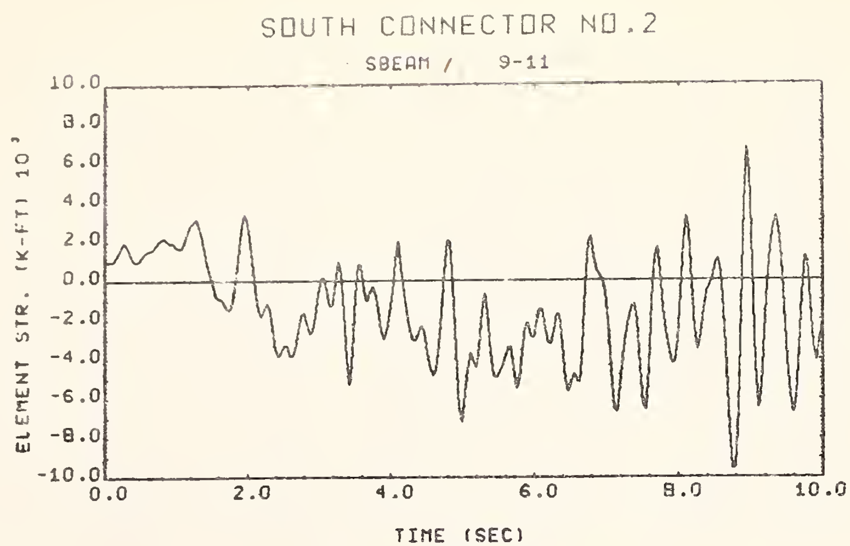


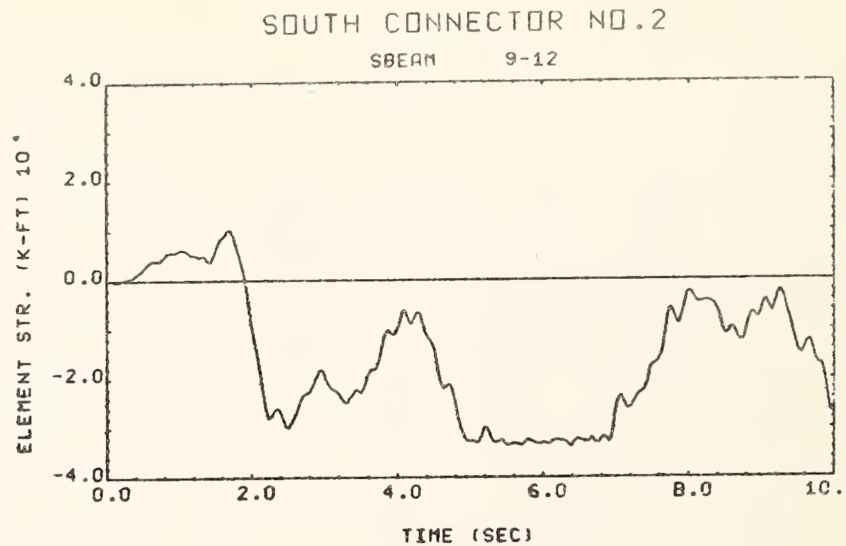
Shear in the Local z - Direction



Moment about the Local z - Axis

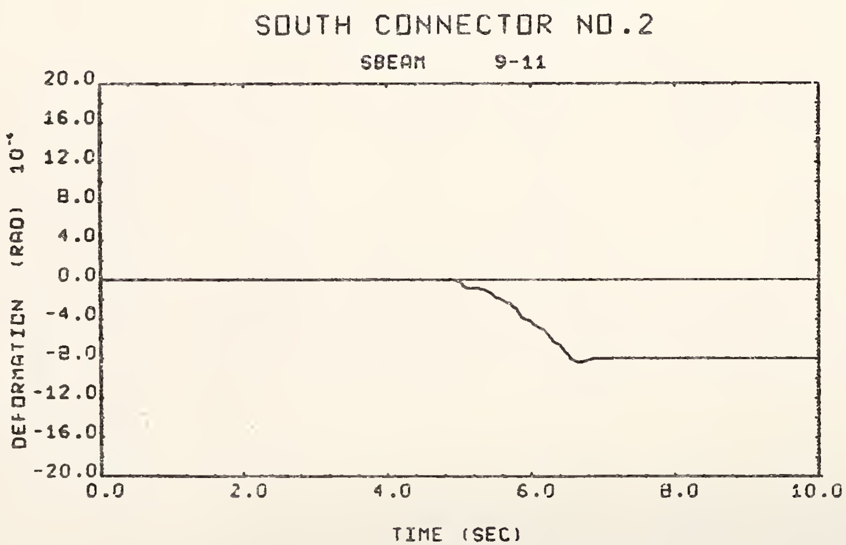
Fig. 37 Generalized Forces in the Girder at the Center of Span # 4





About the Local y - Axis

About the Local z - Axis



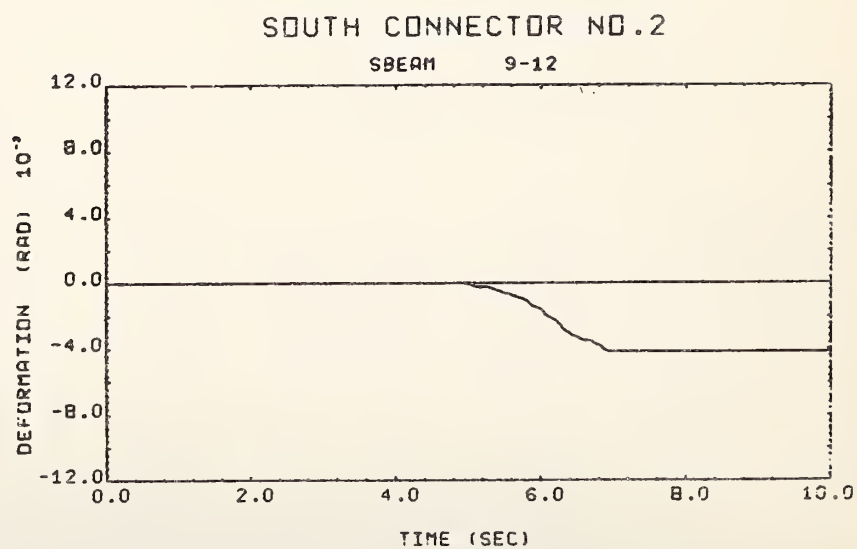


Fig. 38 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3

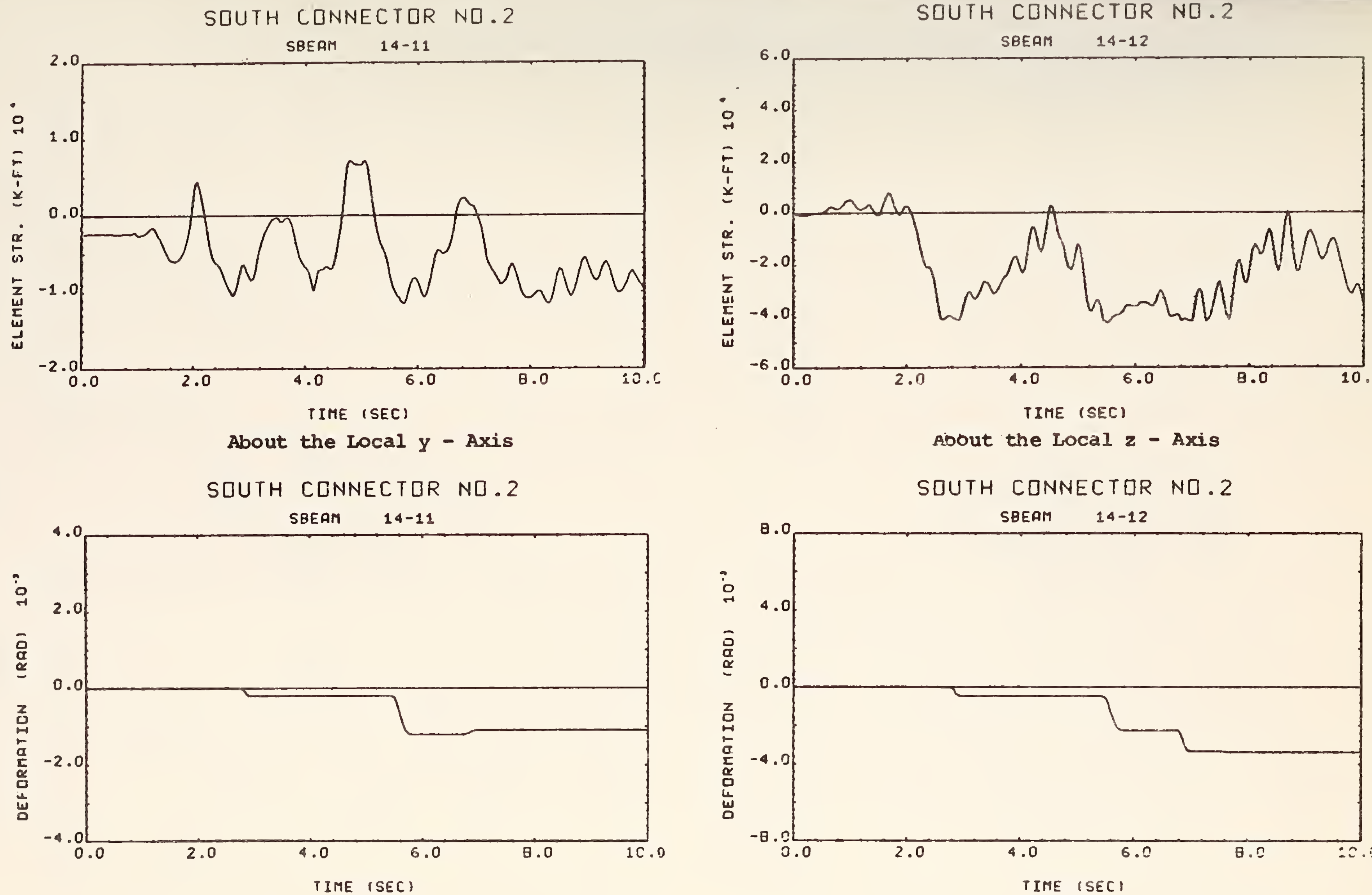


Fig. 39 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

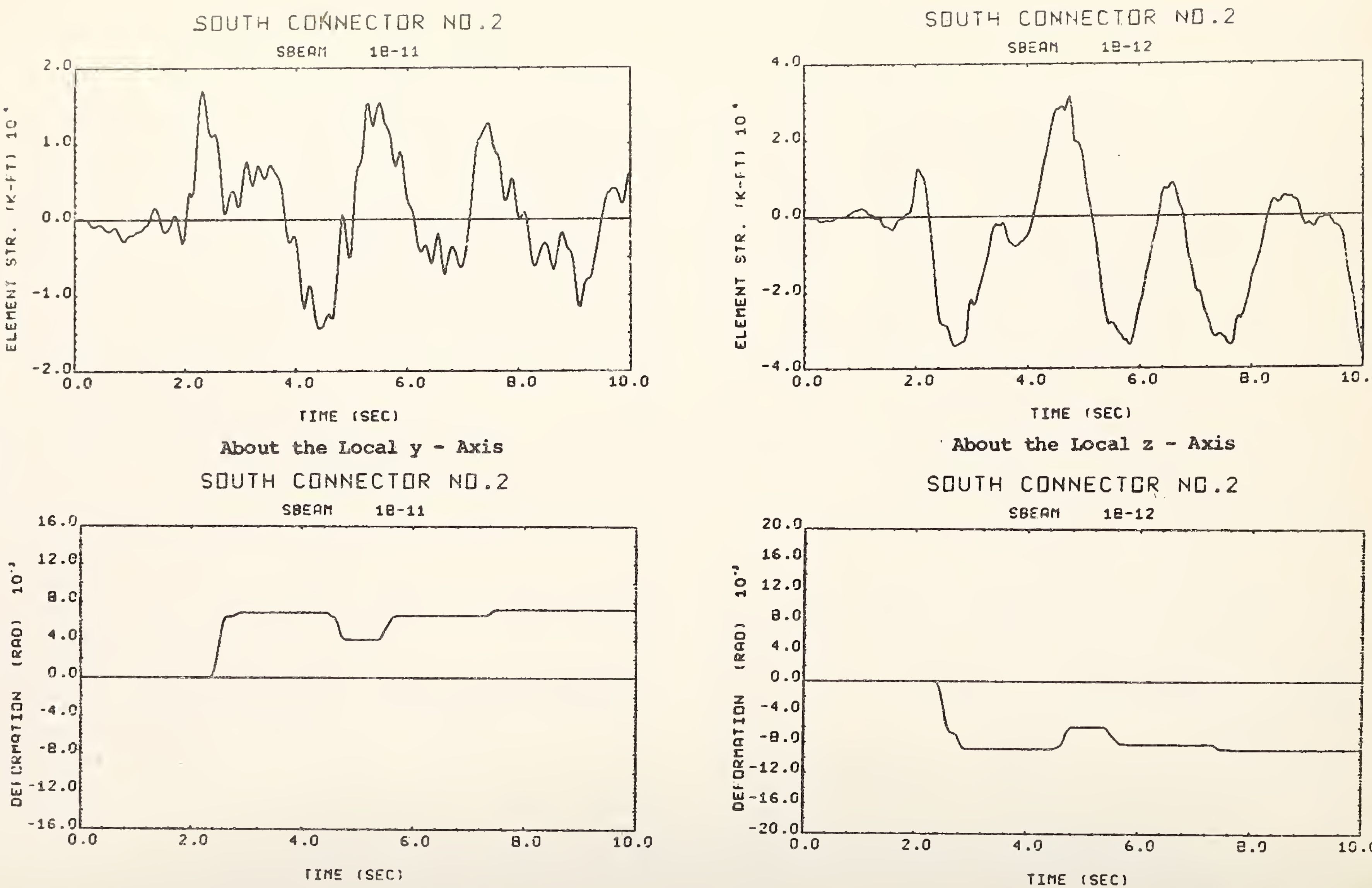
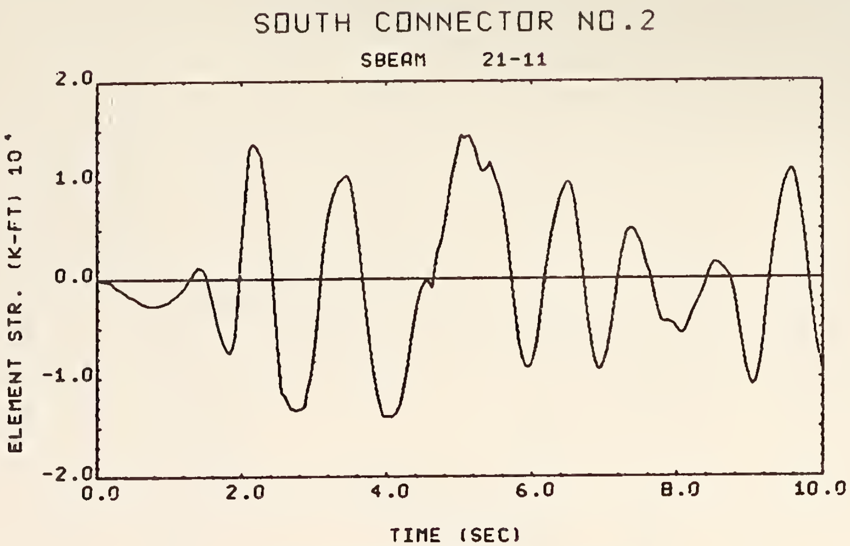
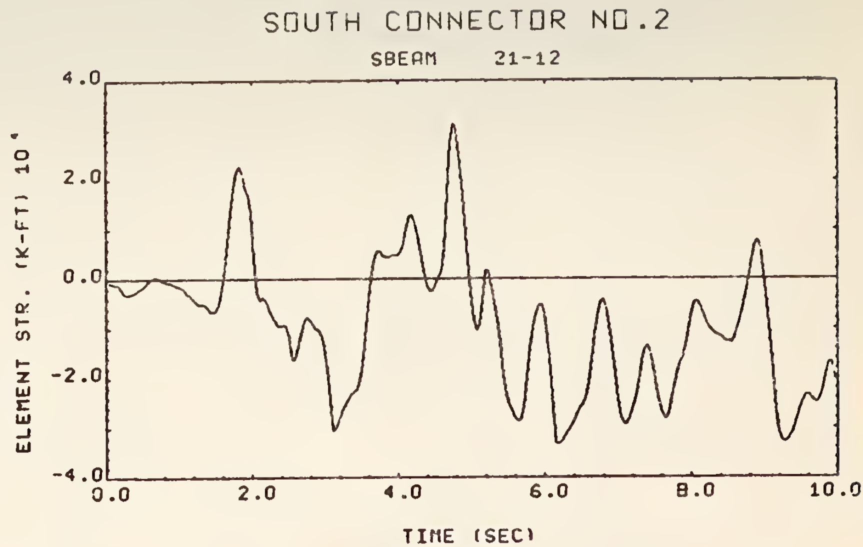


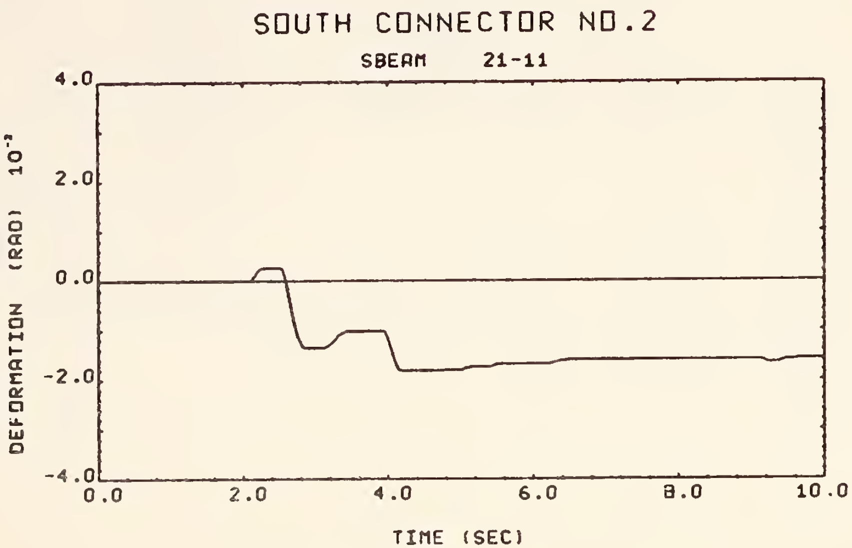
Fig. 40 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 5





About the Local y - Axis

About the Local z - Axis



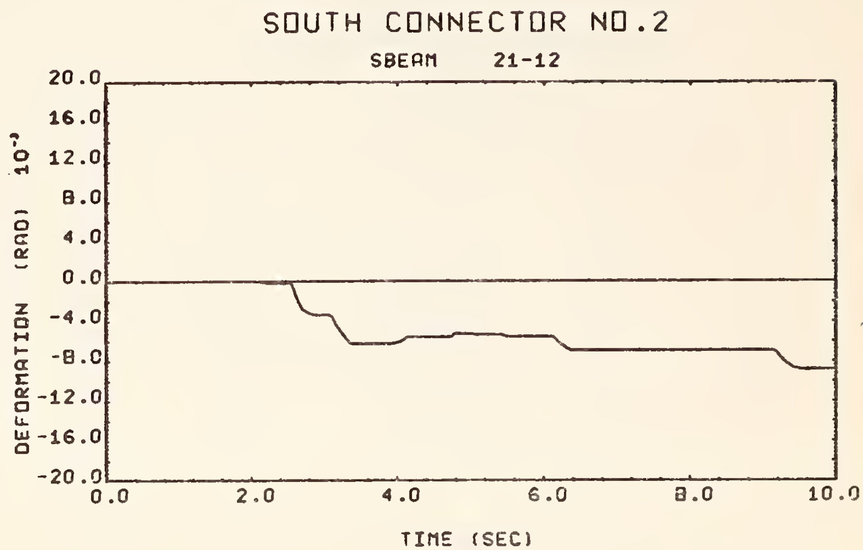
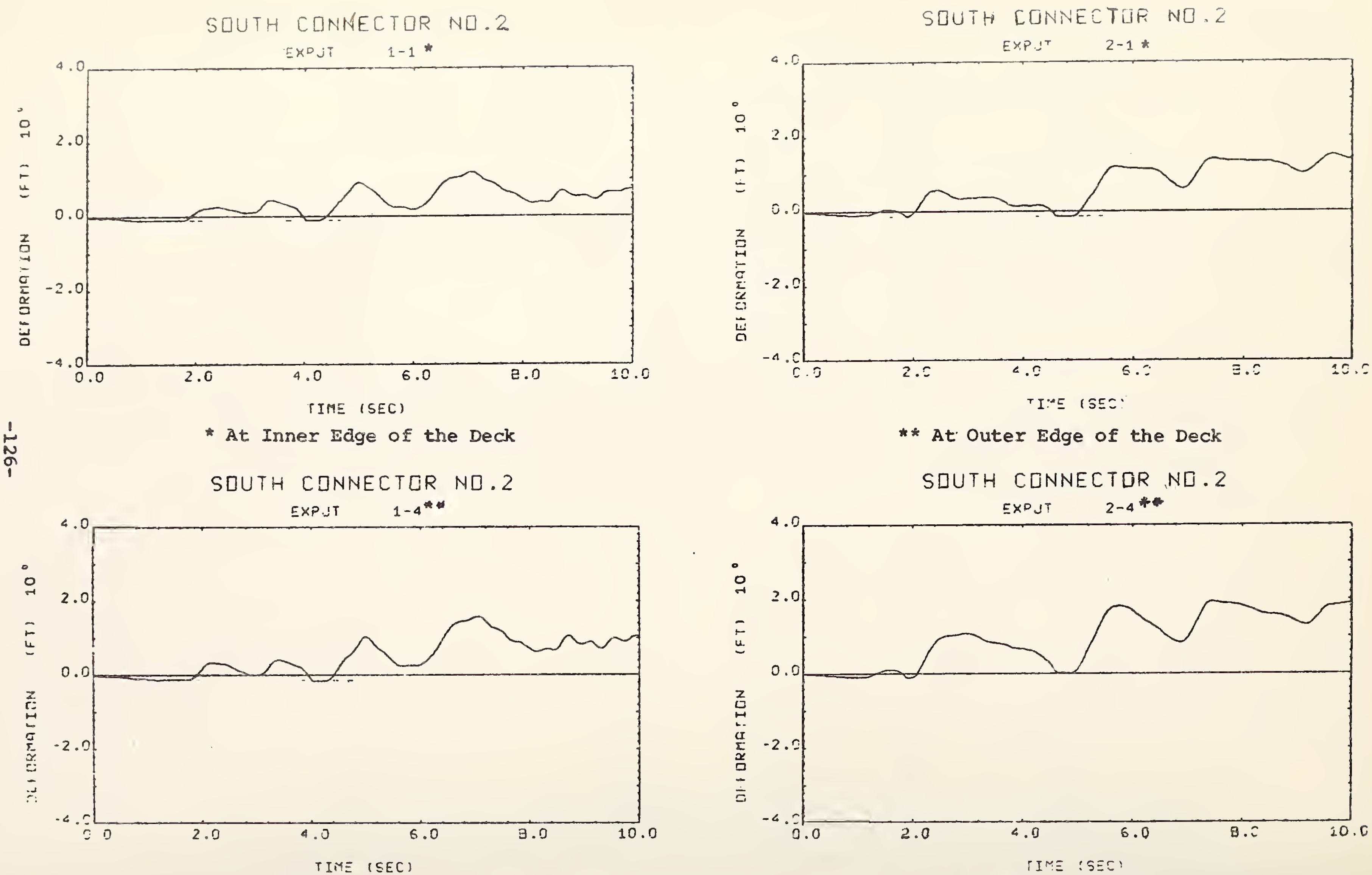


Fig. 41 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 6



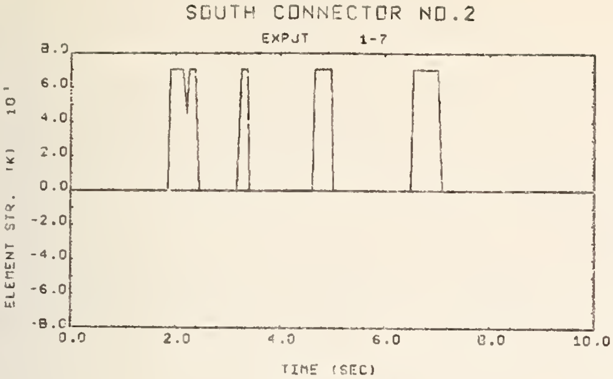
* At Inner Edge of the Deck

** At Outer Edge of the Deck

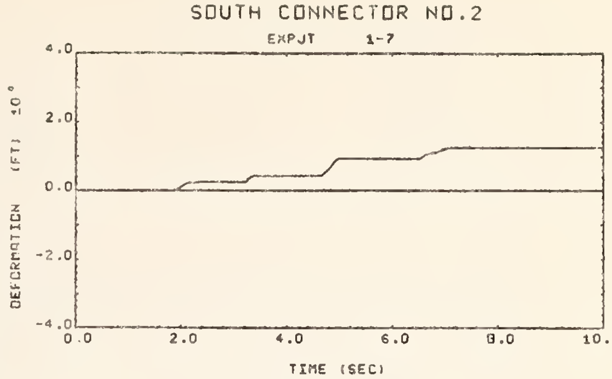
Expansion Joint # 1

Expansion Joint # 2

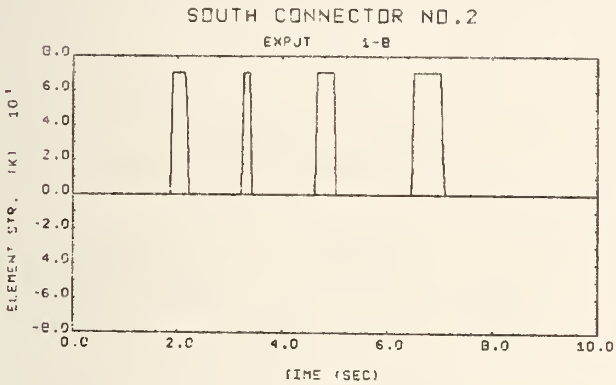
Fig. 42 Longitudinal Joint Separations at Expansion Joints # 1 and # 2



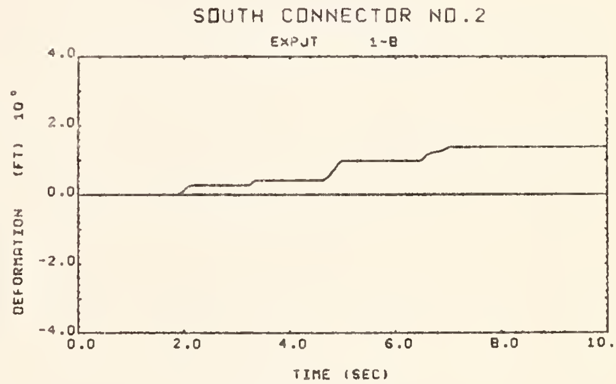
Tie Bar # 1



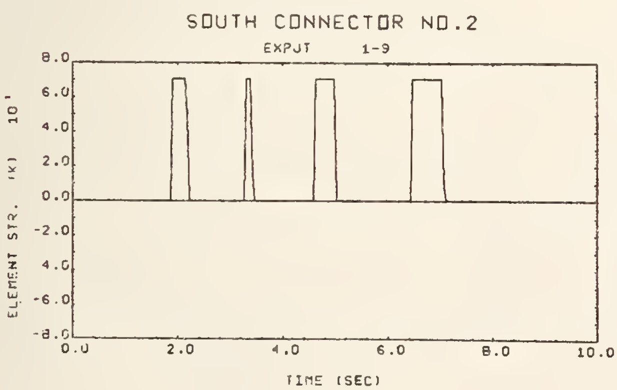
Tie Bar # 1



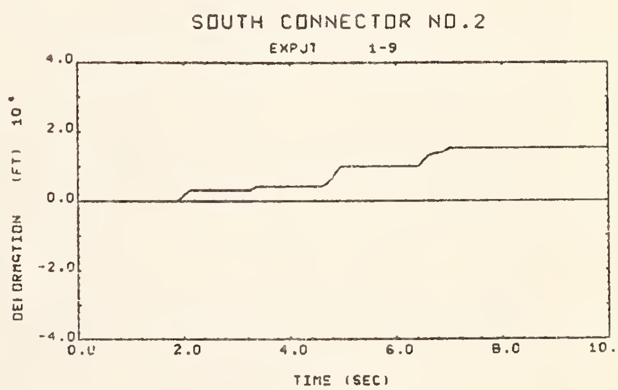
Tie Bar # 2



Tie Bar # 2



Tie Bar # 3



Tie Bar # 3

Fig. 43 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 1

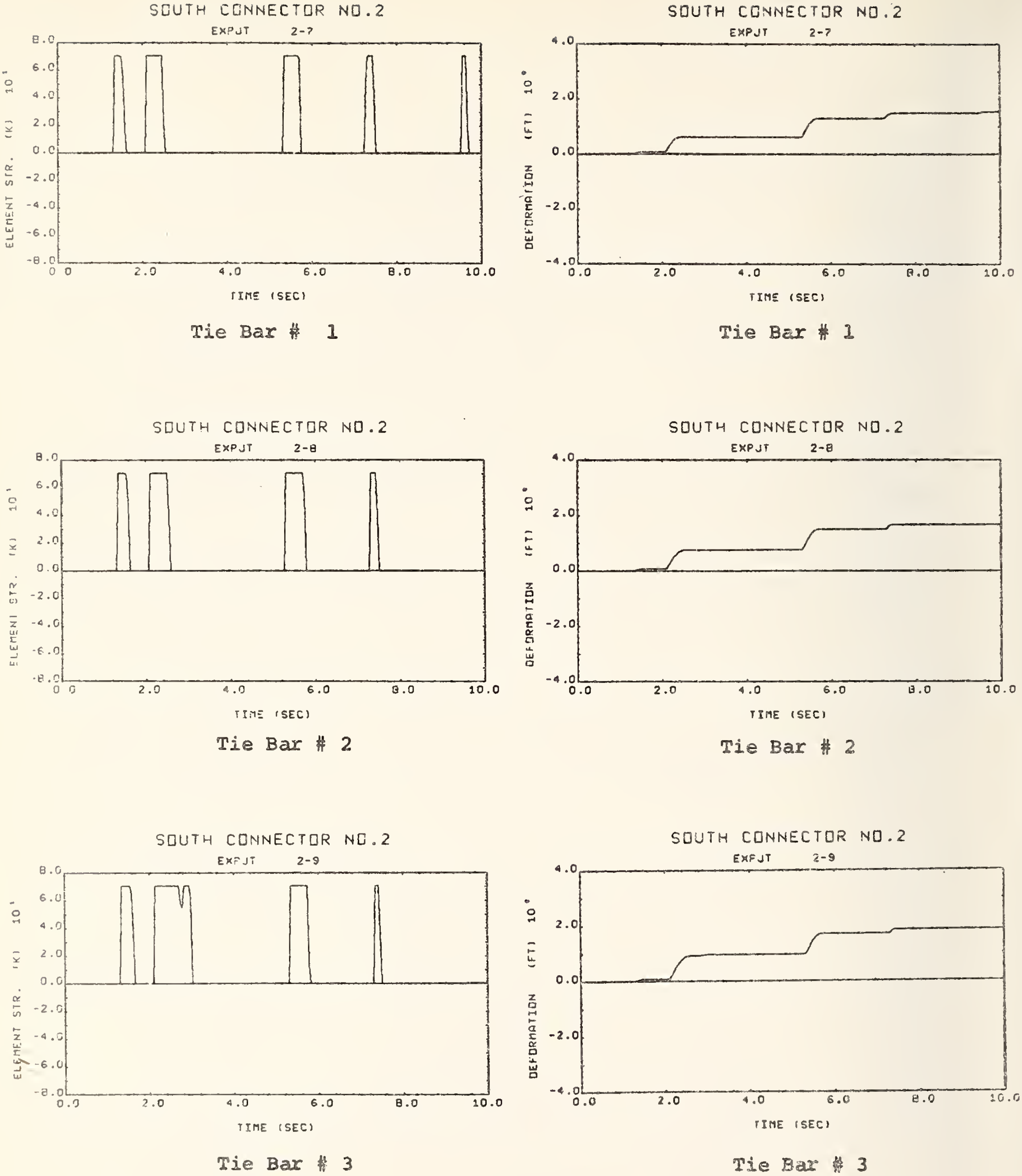
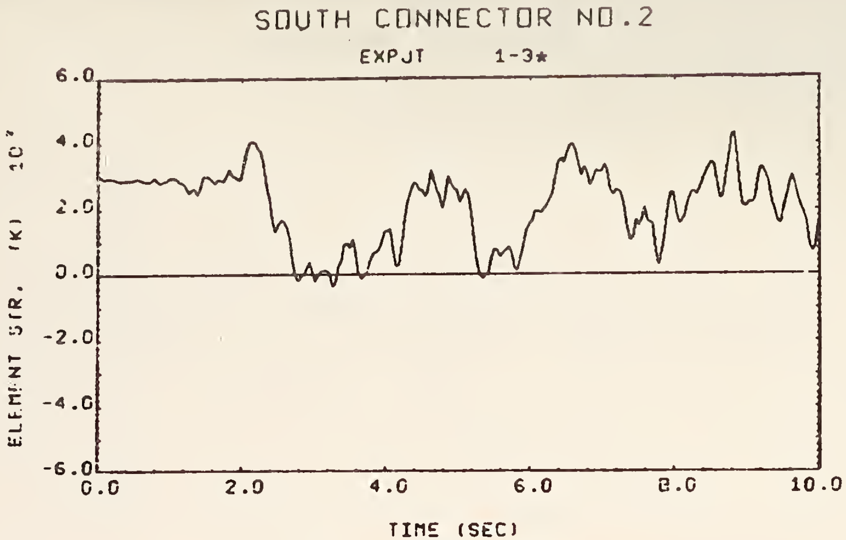
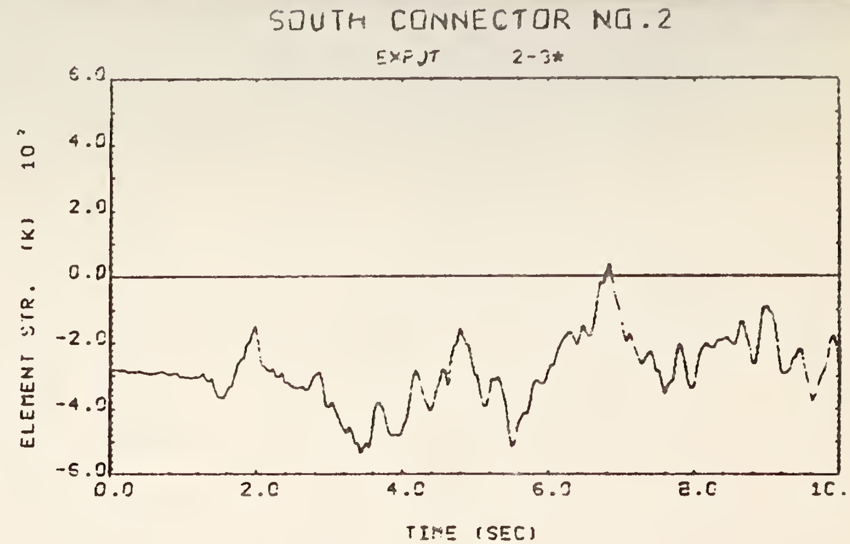


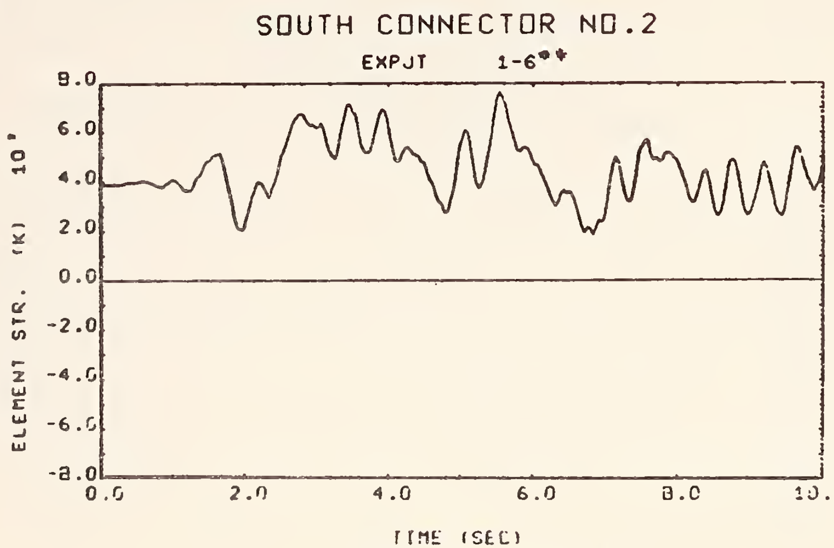
Fig. 44 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 2



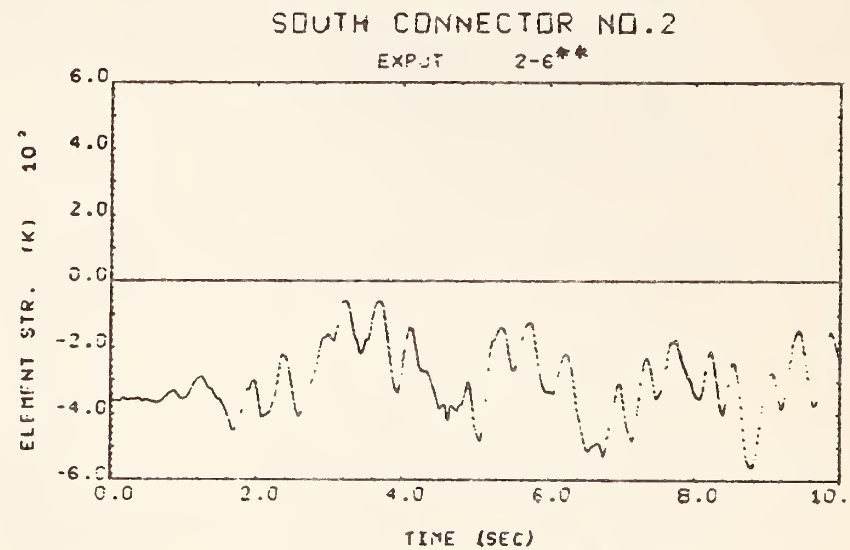
* At Inner Edge of the Deck



** At Outer Edge of the Deck

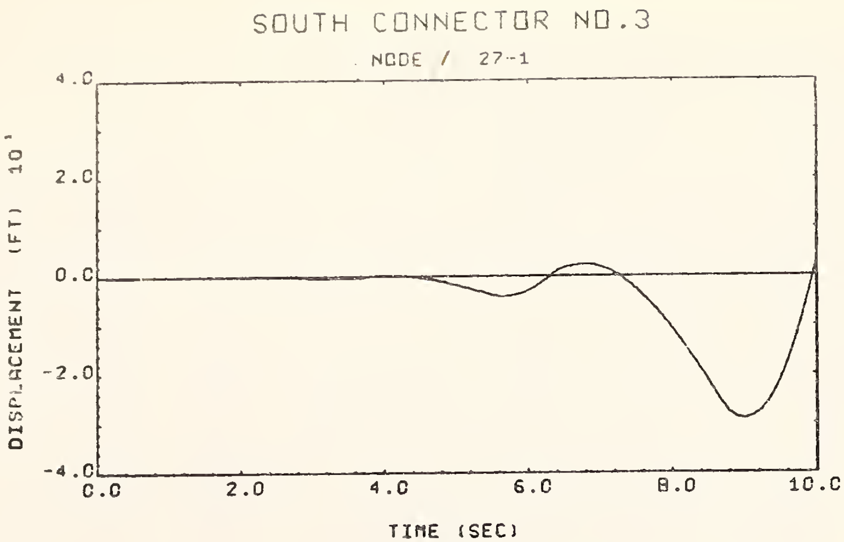


Expansion Joint # 1

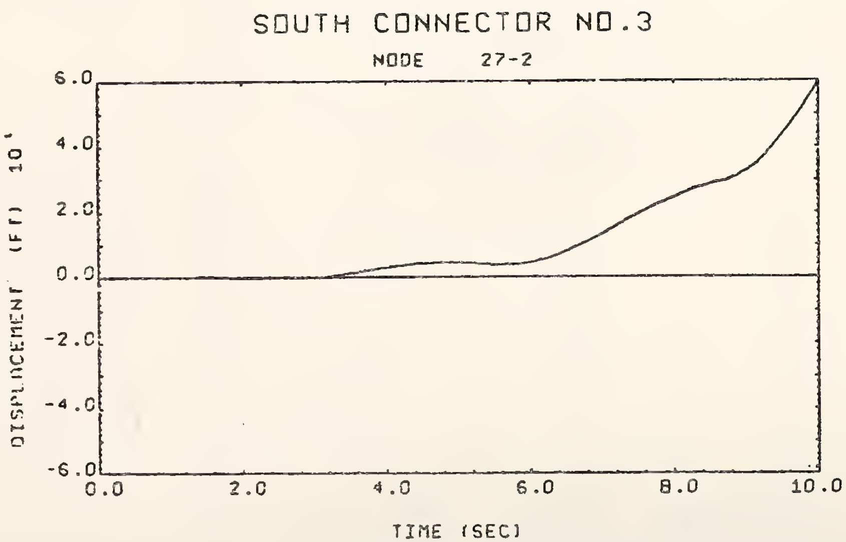


Expansion Joint # 2

Fig. 45 Forces in Vertical Restrainers at Expansion Joint # 1 and # 2

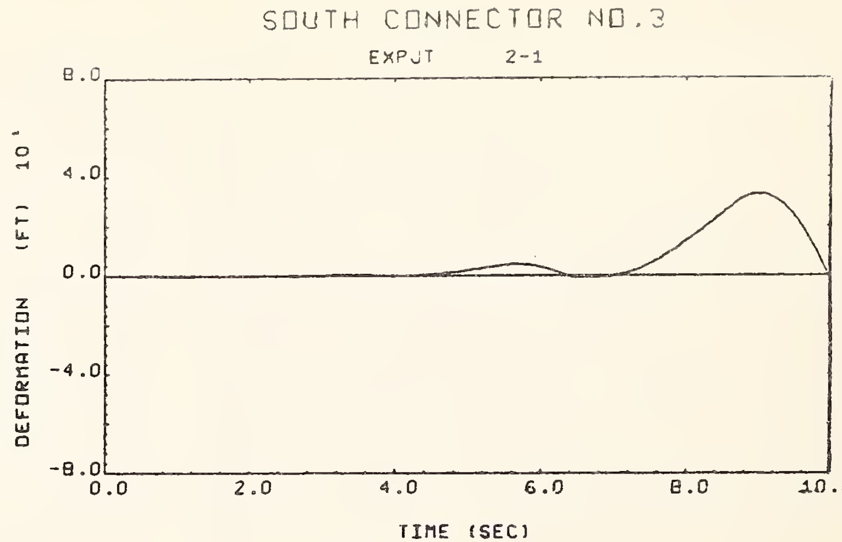


Global X - Component

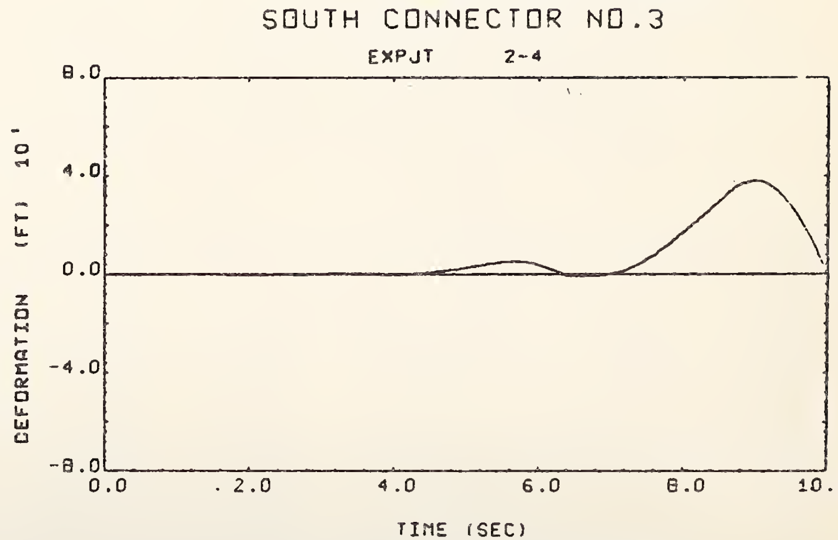


Global Y - Component

Fig. 46 Horizontal Displacements at the Top of Column # 4

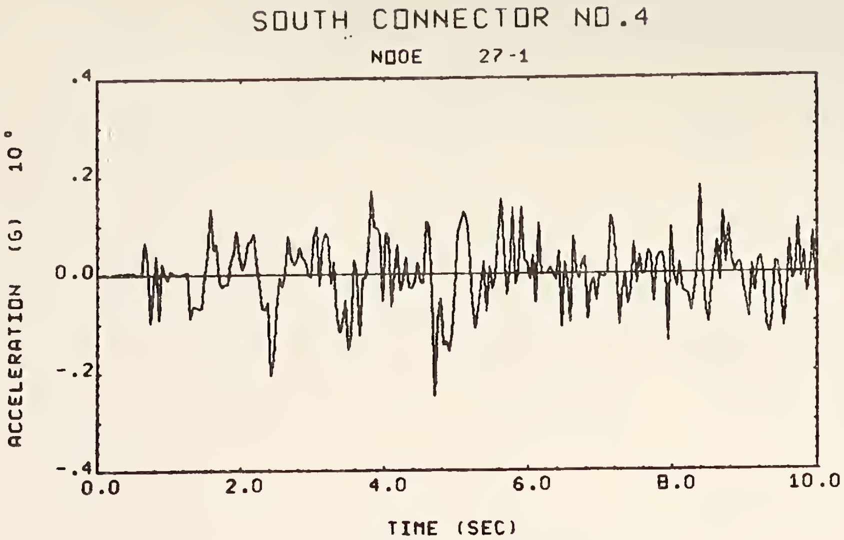


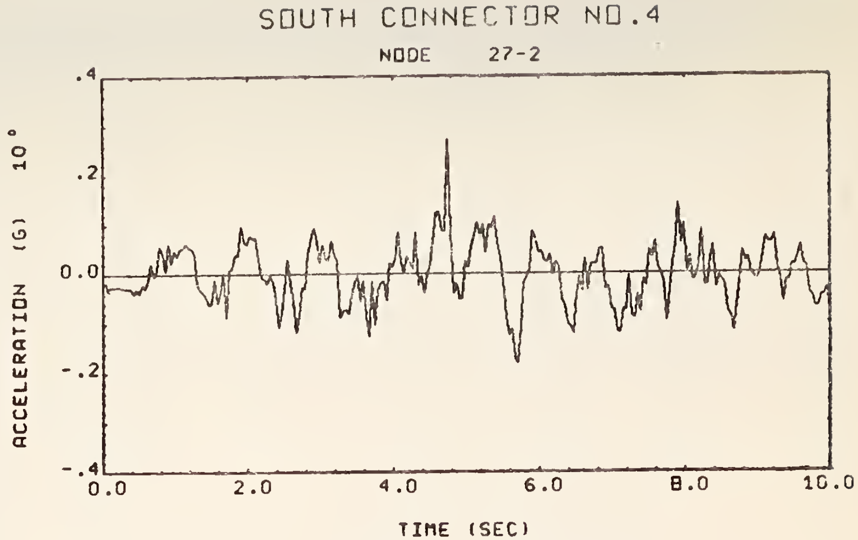
At Inner Edge of the Deck



At Outer Edge of the Deck

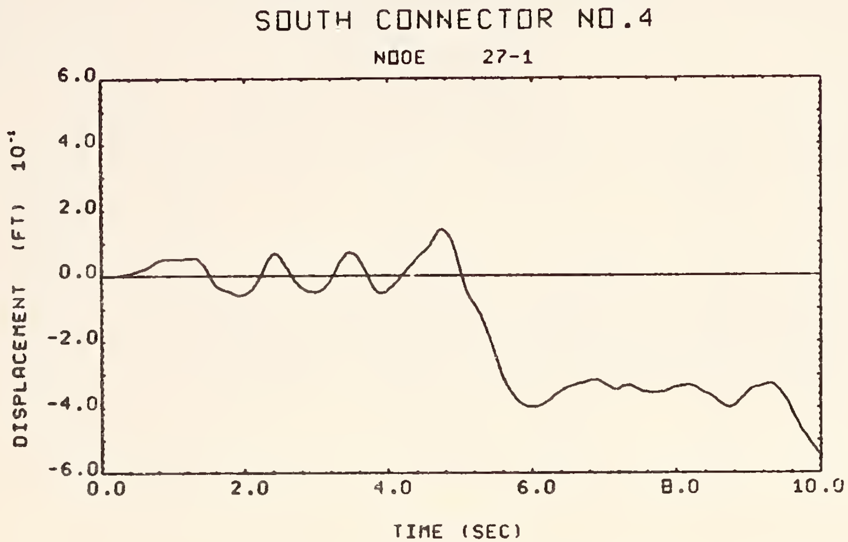
Fig. 47 Longitudinal Joint Separations at Expansion Joint # 2





Global X - Component

Global Y - Component



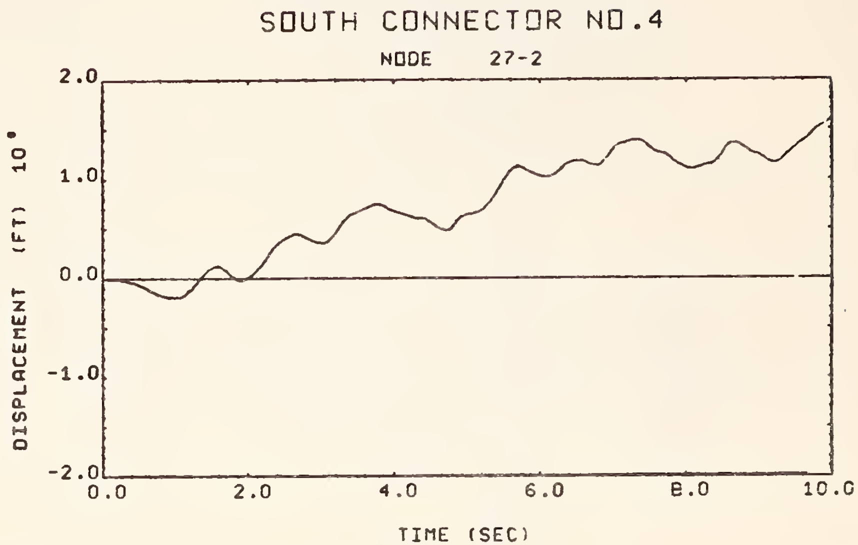


Fig. 48 Horizontal Accelerations and Displacements at the Top of Column # 4

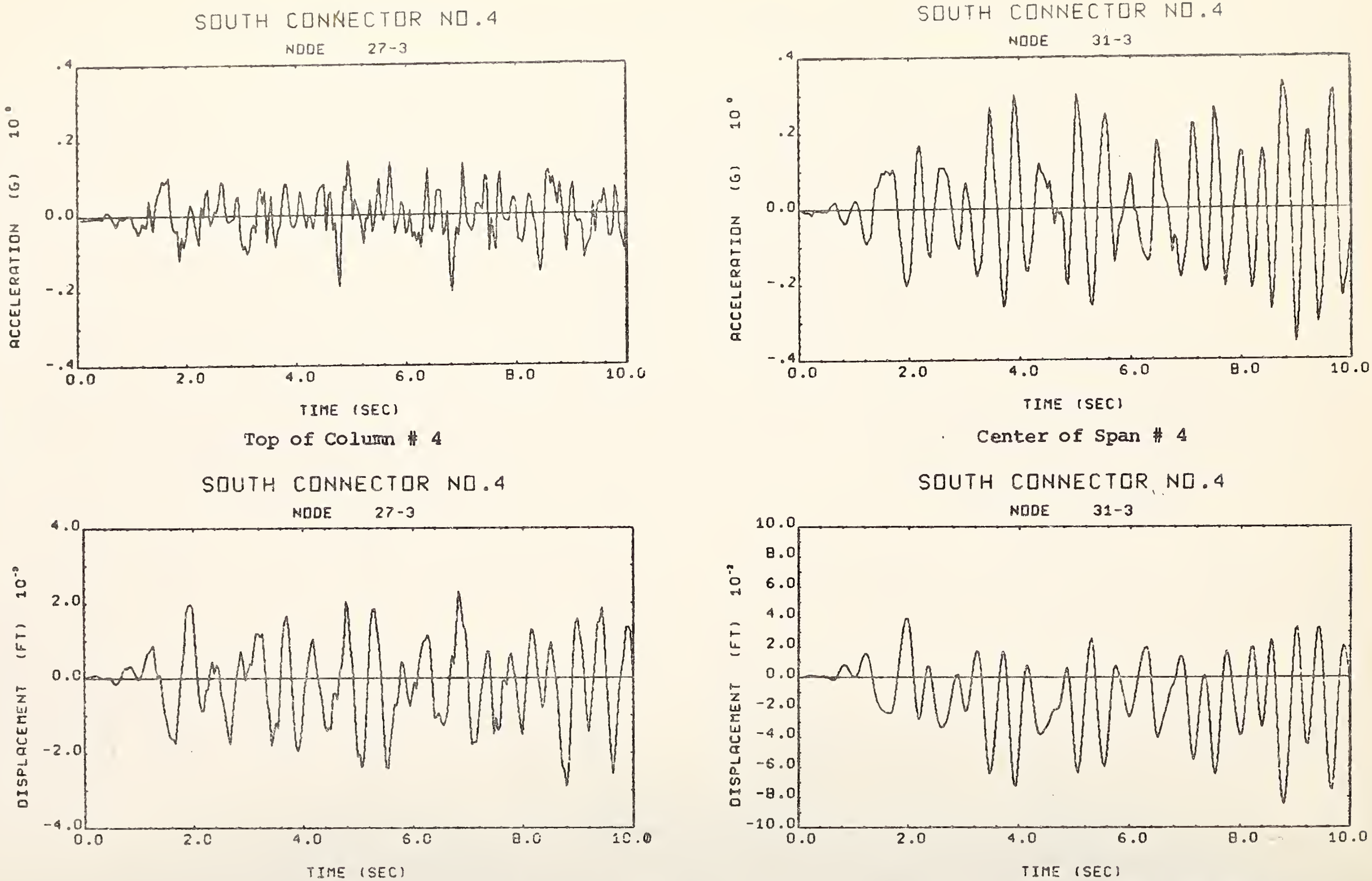
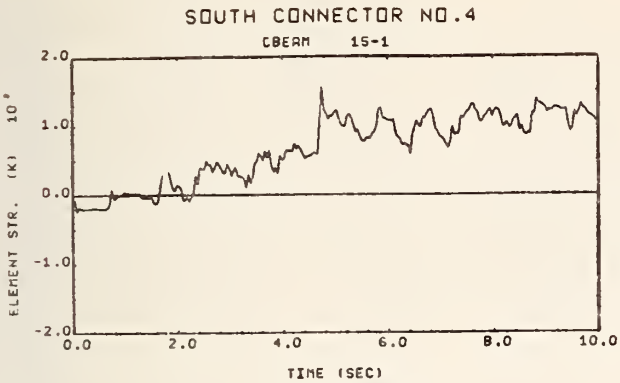
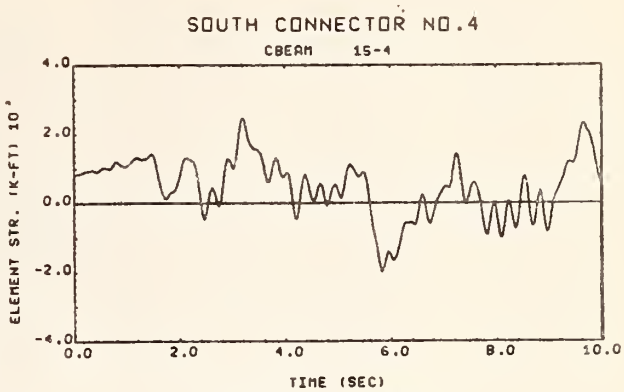


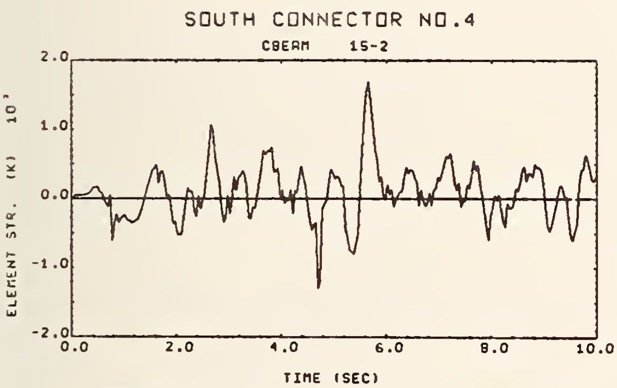
Fig. 49 Vertical Accelerations and Displacements at the Top of Column # 4 and the Center of Span # 4



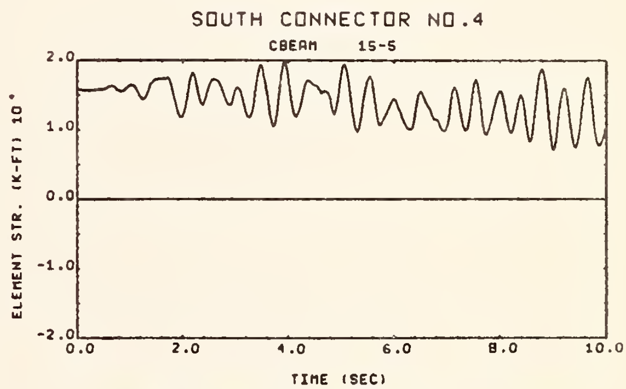
Axial Force



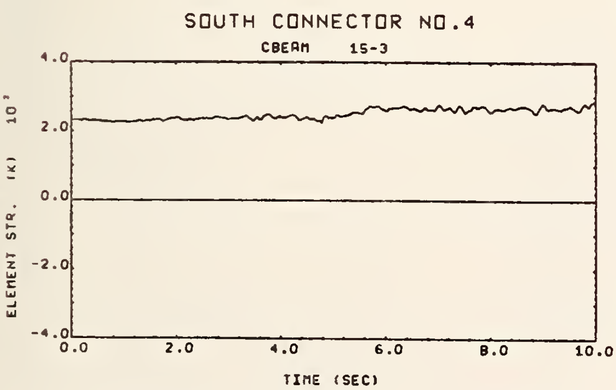
Torsional Moment



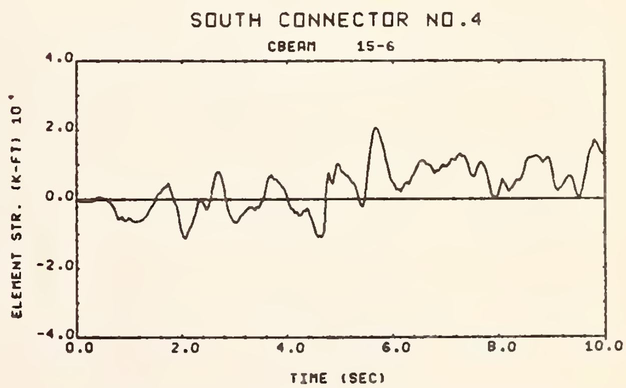
Shear in the Local y - Direction



Moment about the Local y - Axis

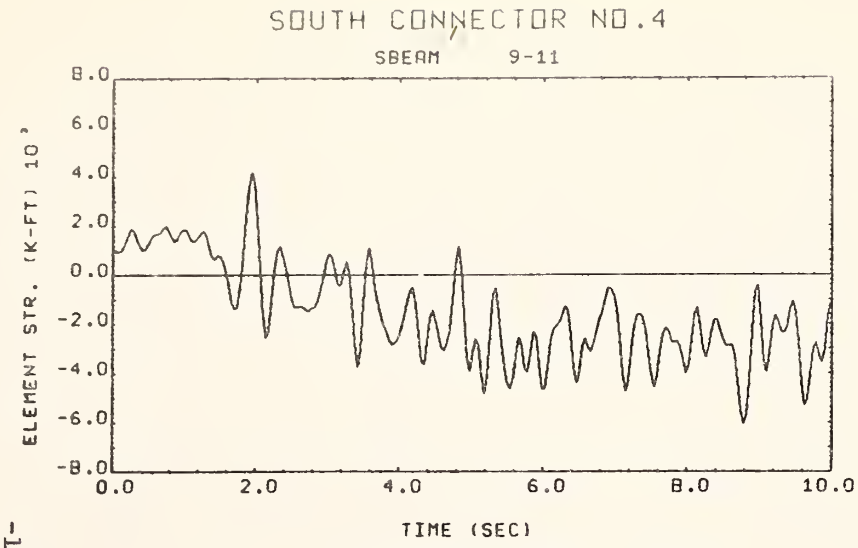


Shear in the Local z - Direction

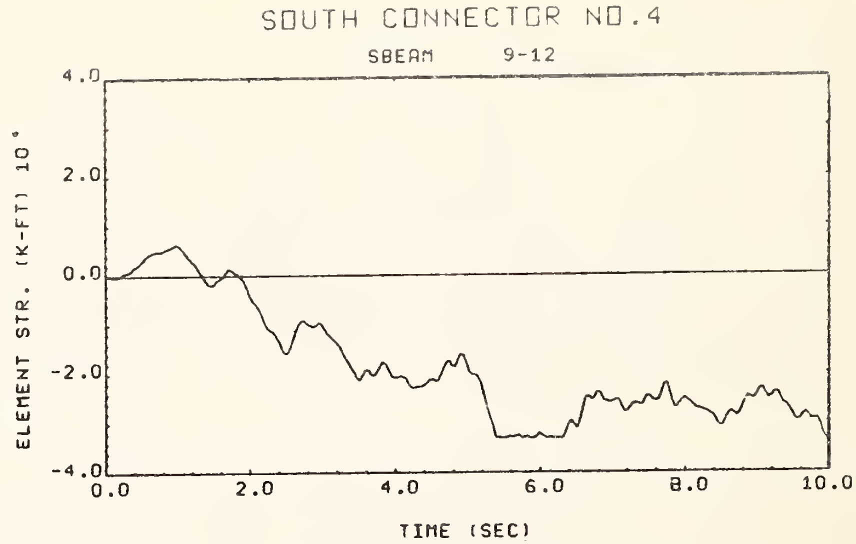


Moment about the Local z - Axis

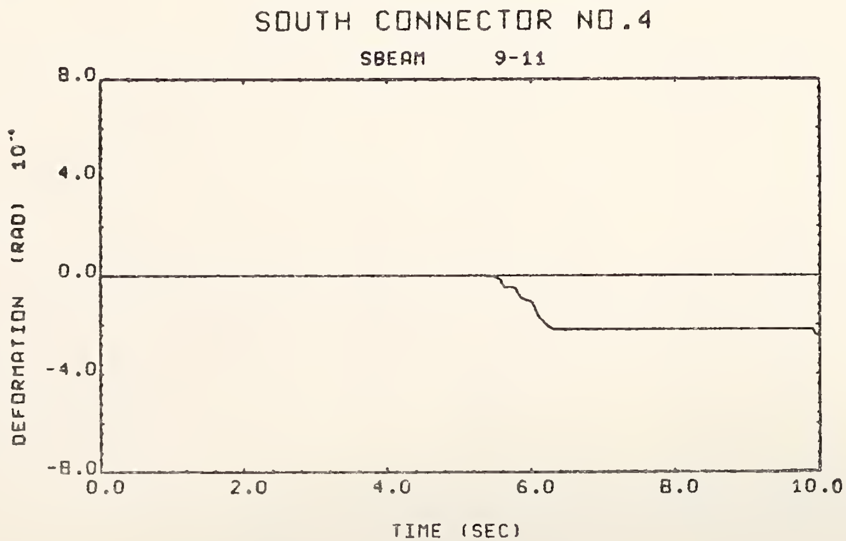
Fig. 50 Generalized Forces in the Girder at the Center of Span # 4



About the Local y - Axis



About the Local z - Axis



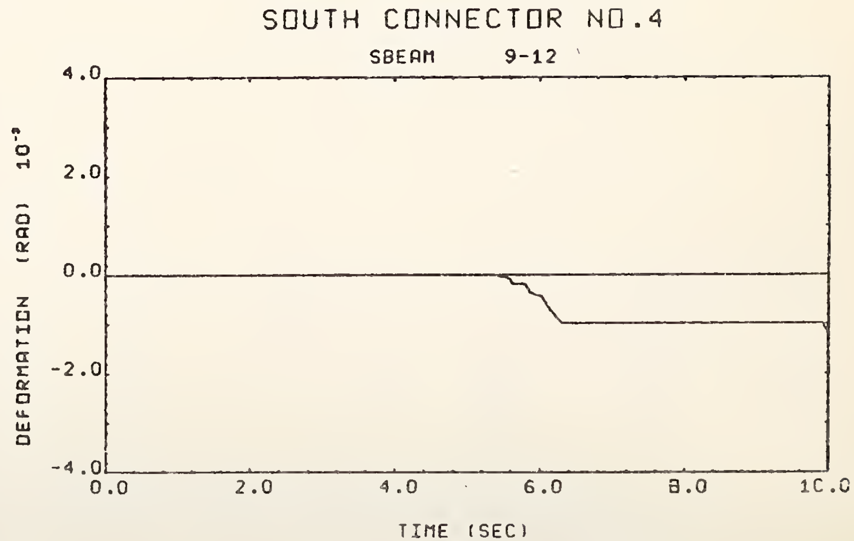


Fig. 51 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3

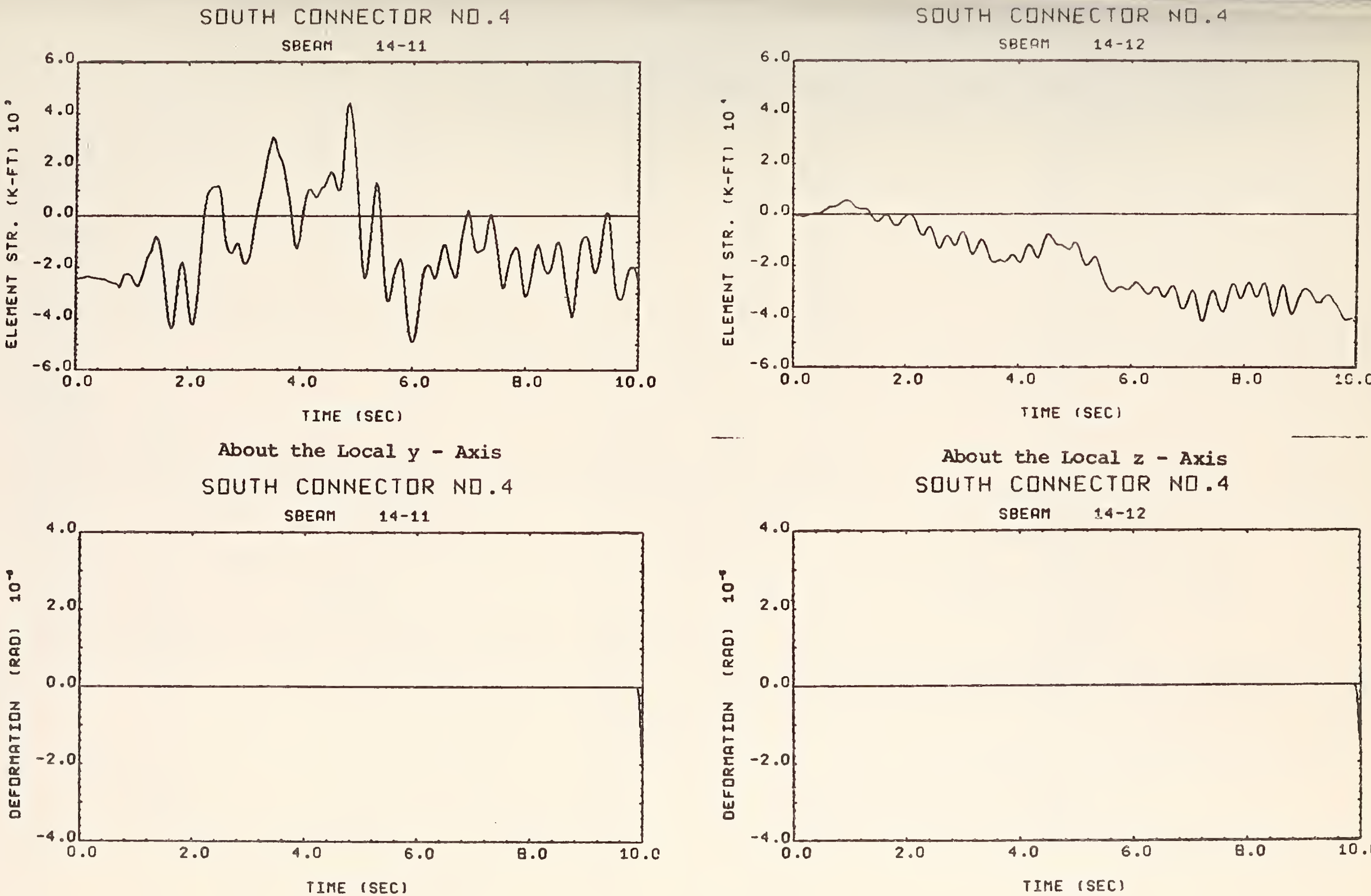


Fig. 52 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

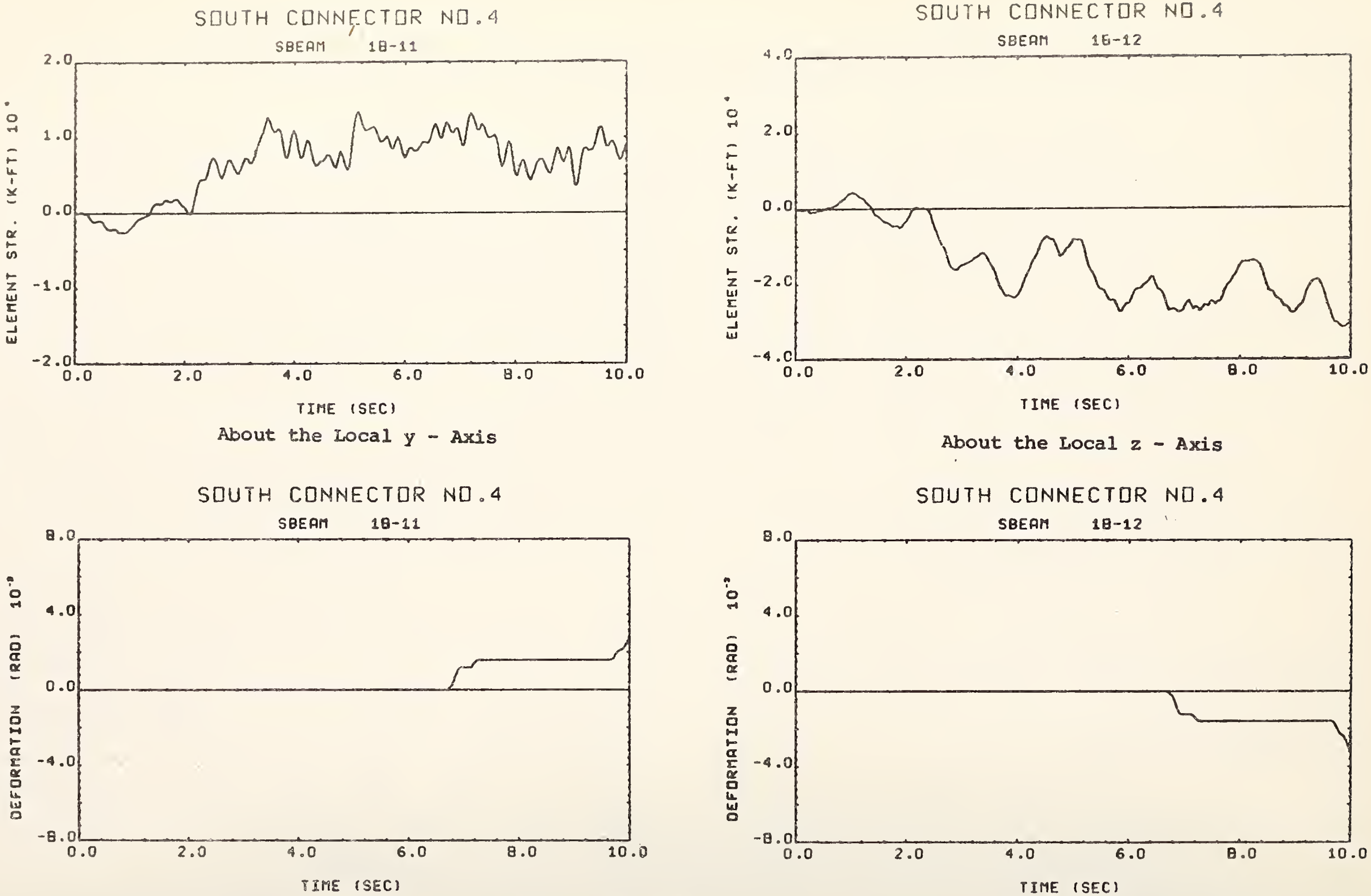
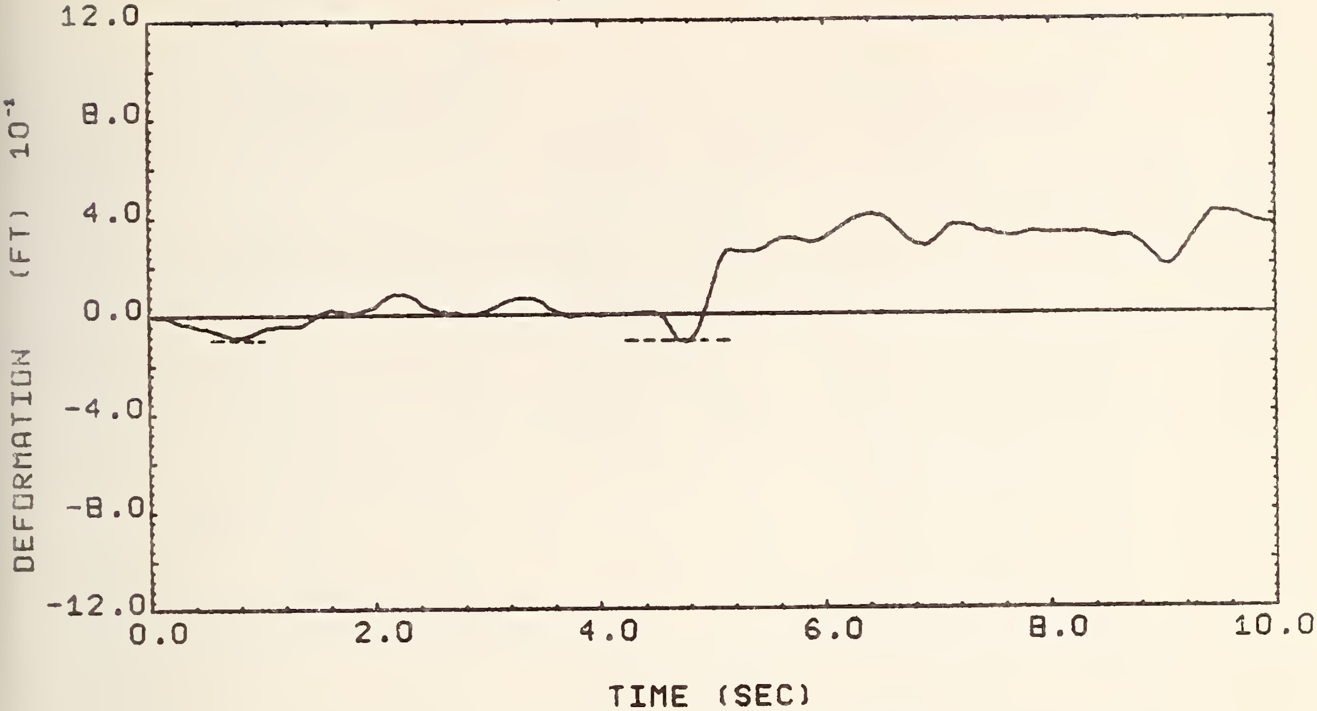


Fig. 53 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 5

SOUTH CONNECTOR NO.4

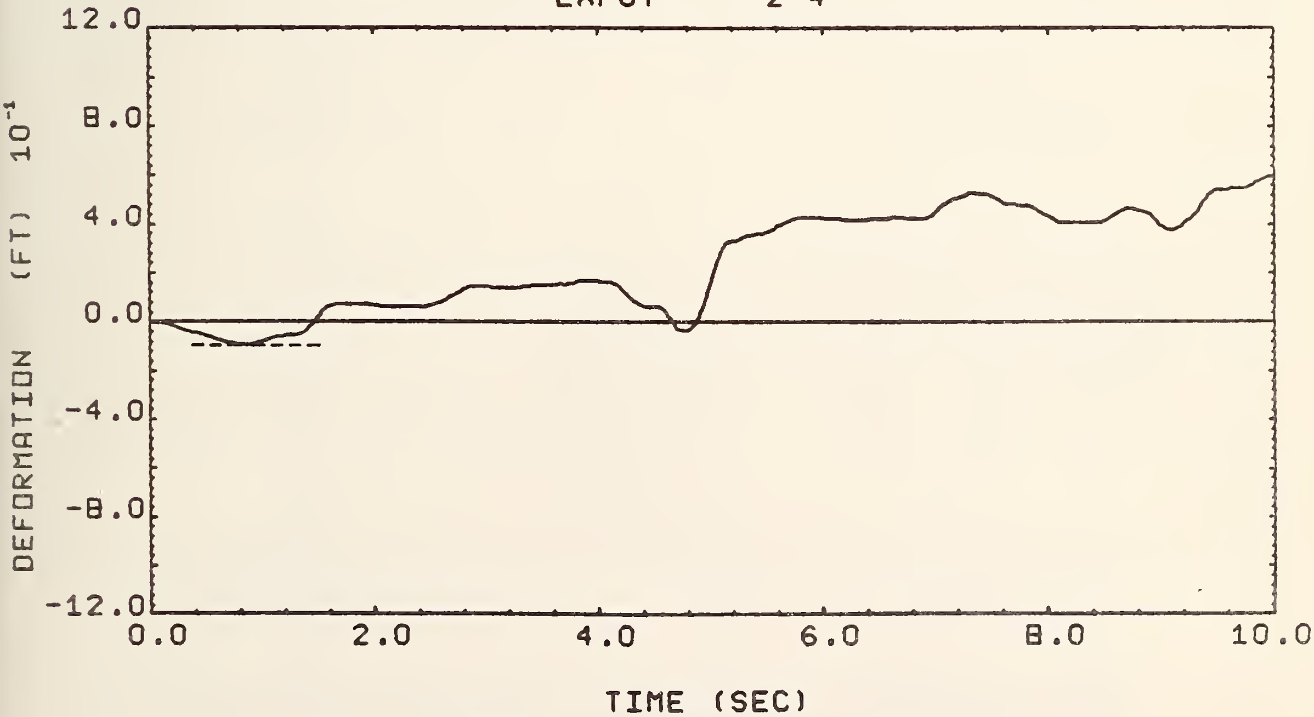
EXPJT 2-1



At Inner Edge of the Deck

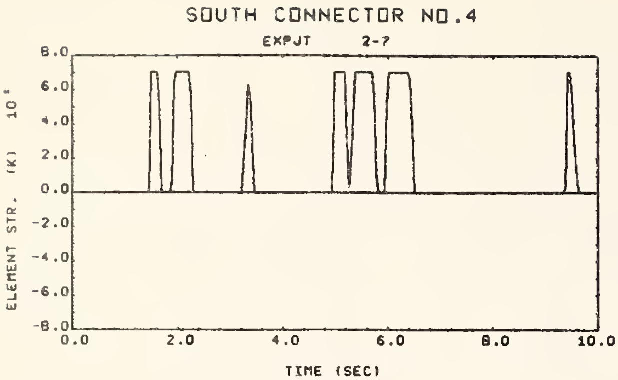
SOUTH CONNECTOR NO.4

EXPJT 2-4

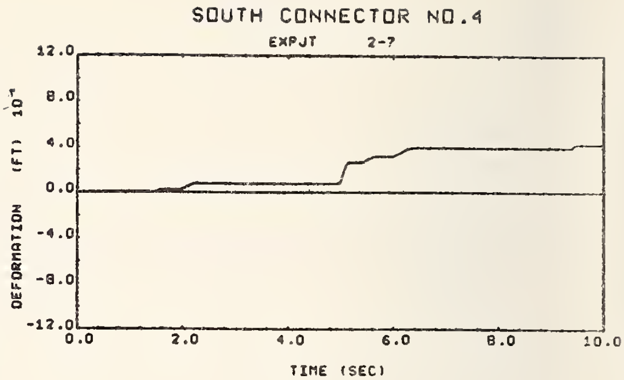


At Outer Edge of the Deck

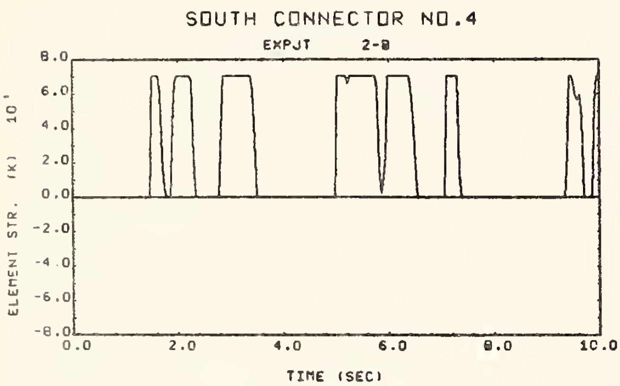
Fig. 54 Longitudinal Joint Separations at Expansion Joint # 2



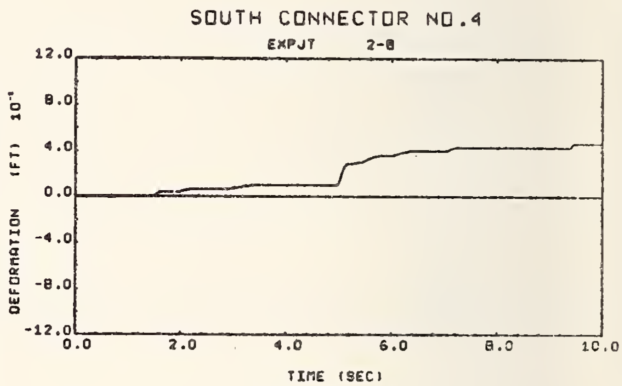
Tie Bar # 1



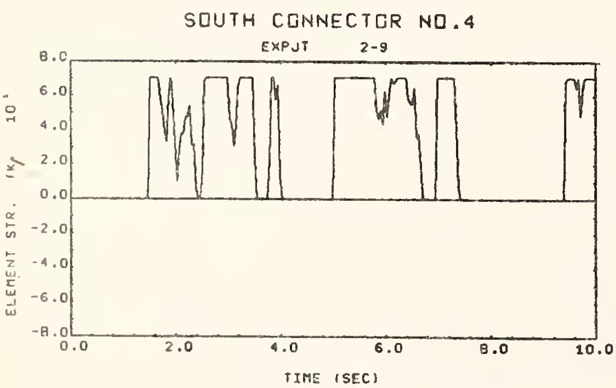
Tie Bar # 1



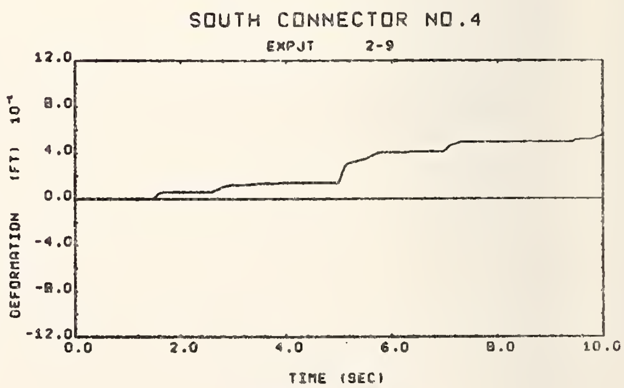
Tie Bar # 2



Tie Bar # 2



Tie Bar # 3



Tie Bar # 3

Fig. 55 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 2

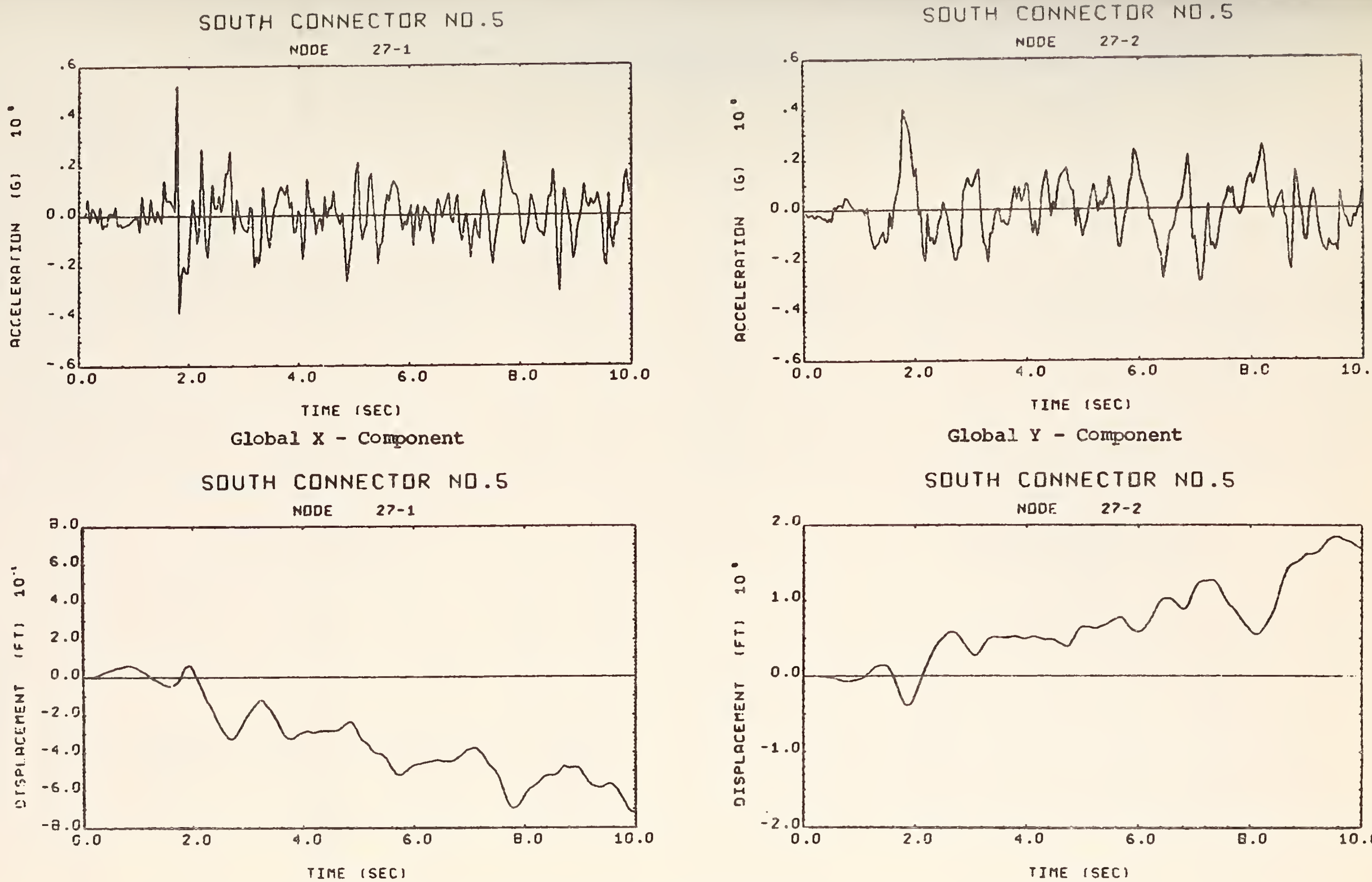


Fig. 56 Horizontal Accelerations and Displacements at the Top of Column # 4

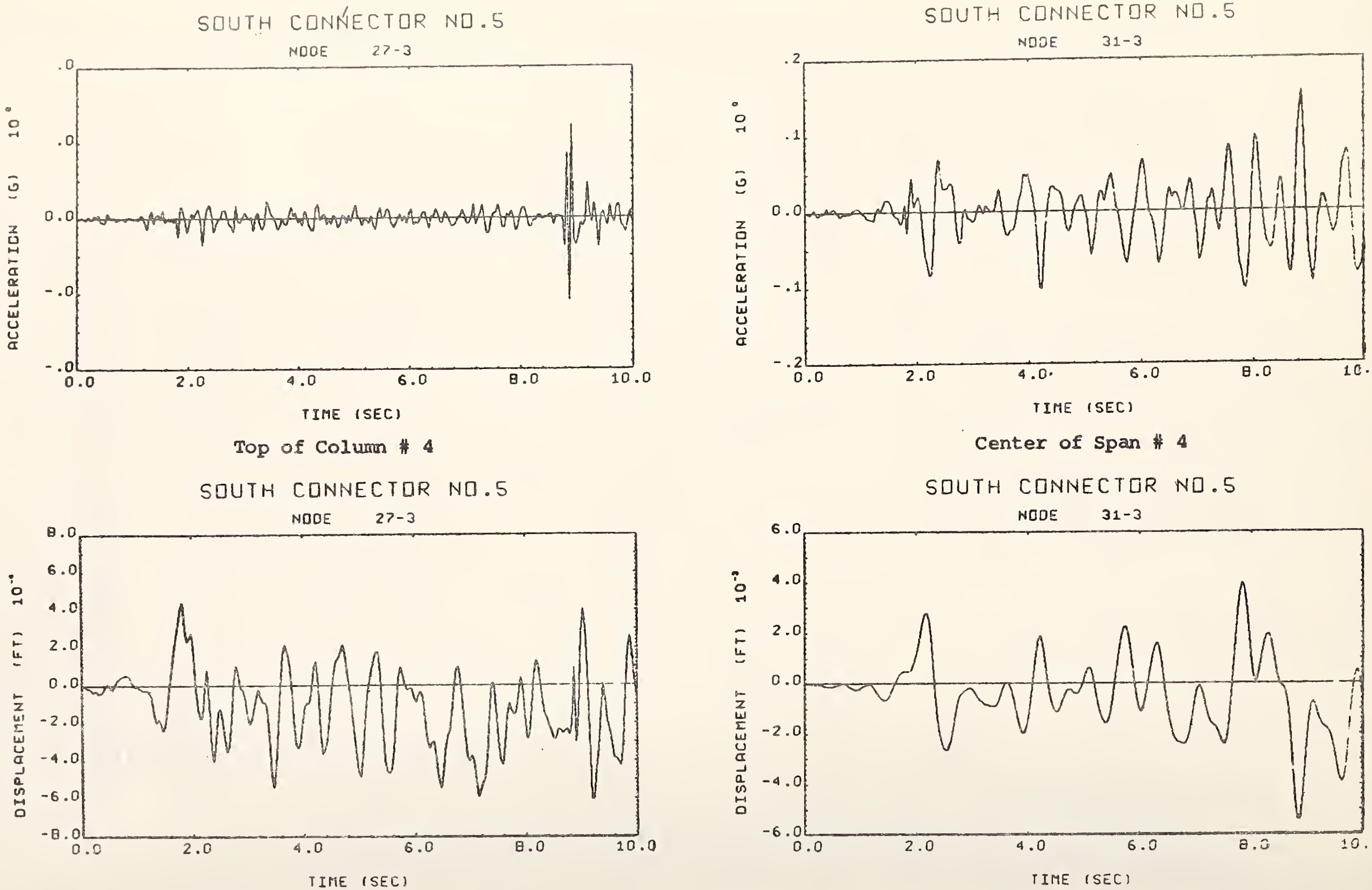
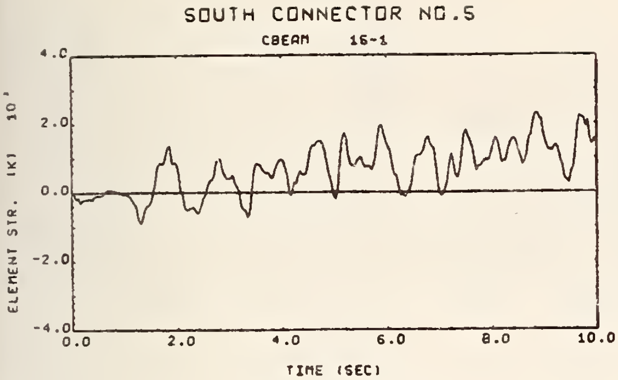
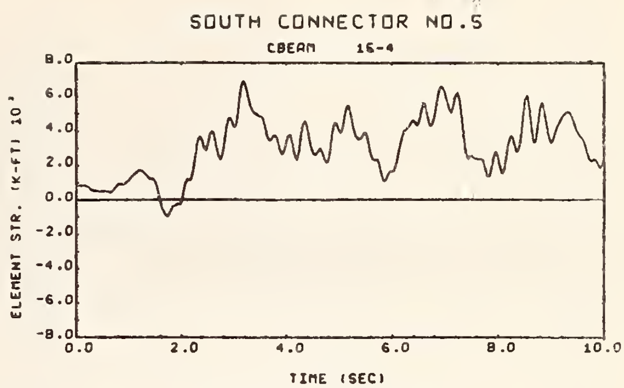


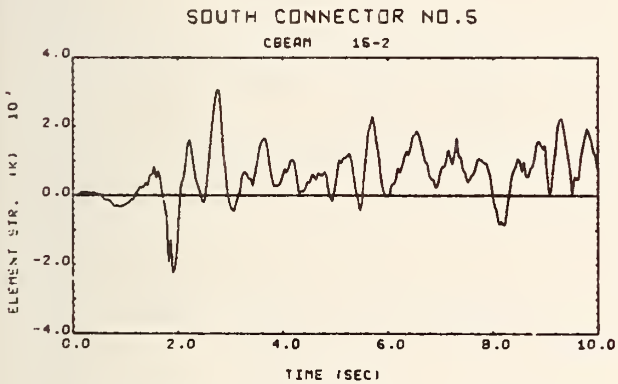
Fig. 57 Vertical Accelerations and Displacements at the Top of Column # 4 and the Center of Span # 4



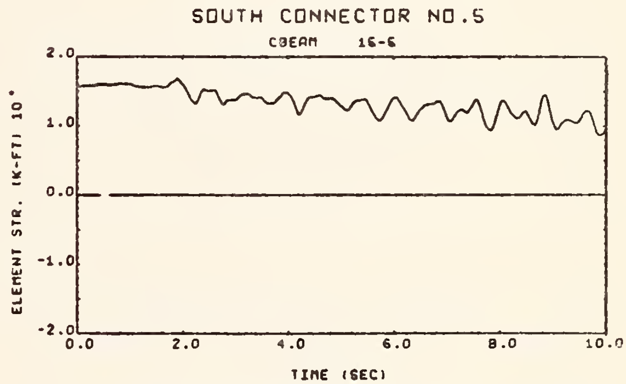
Axial Force



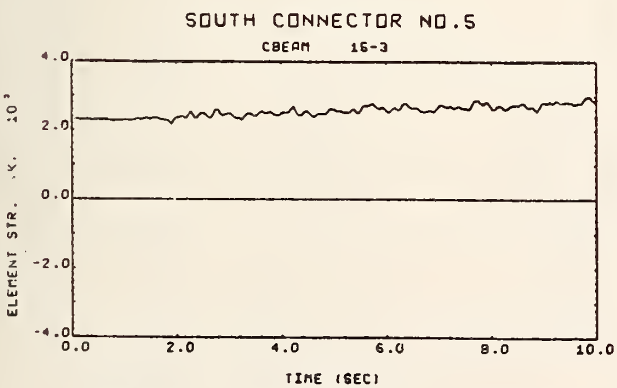
Torsional Moment



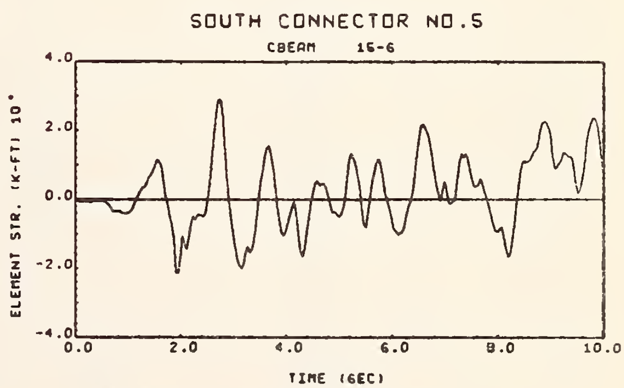
Shear in the Local y - Direction



Moment about the Local y - Axis



Shear in the Local z - Direction



Moment about the Local z - Axis

Fig. 58 Generalized Forces in the Girder at the Center of Span # 4

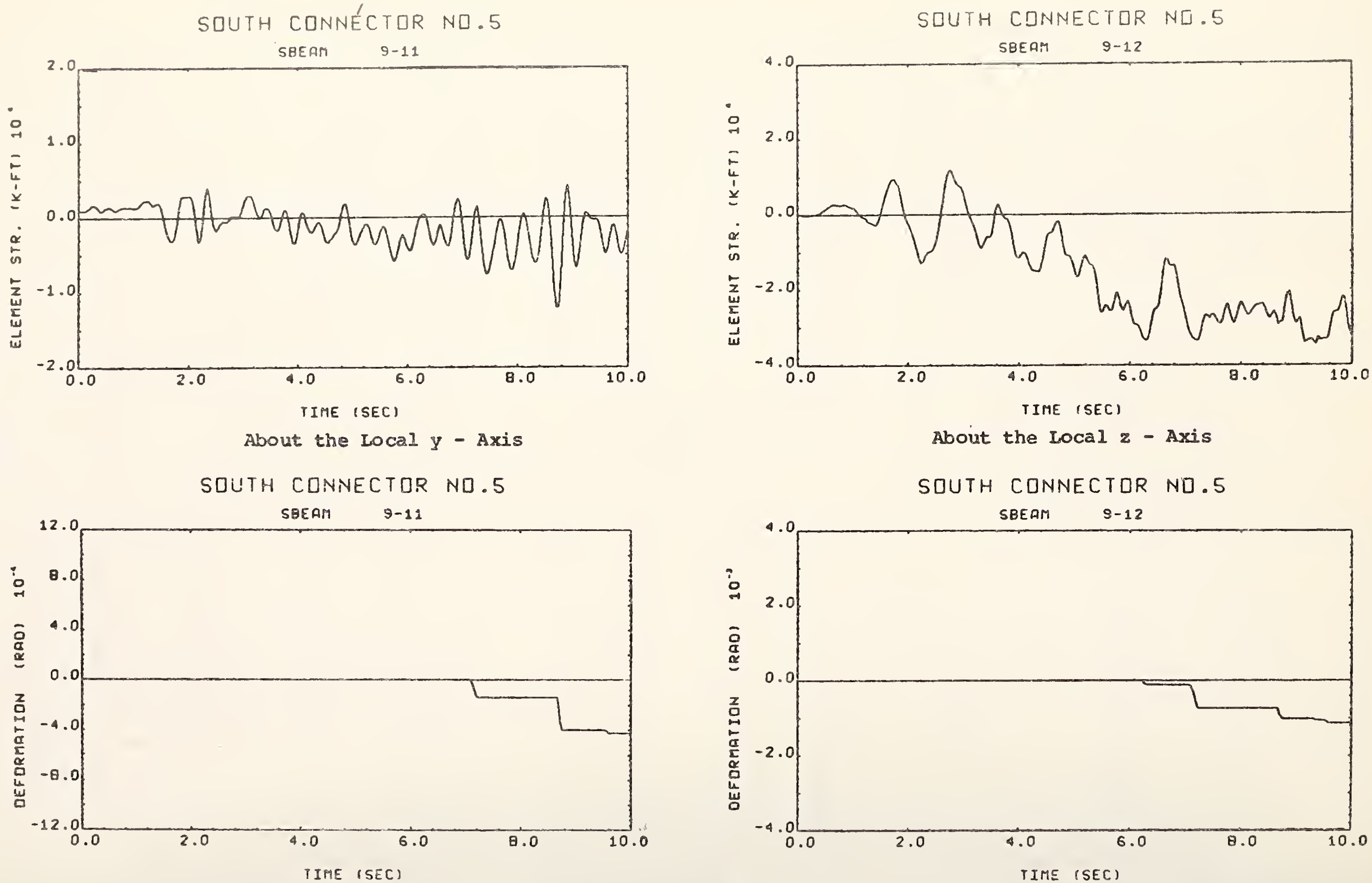


Fig. 59 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3

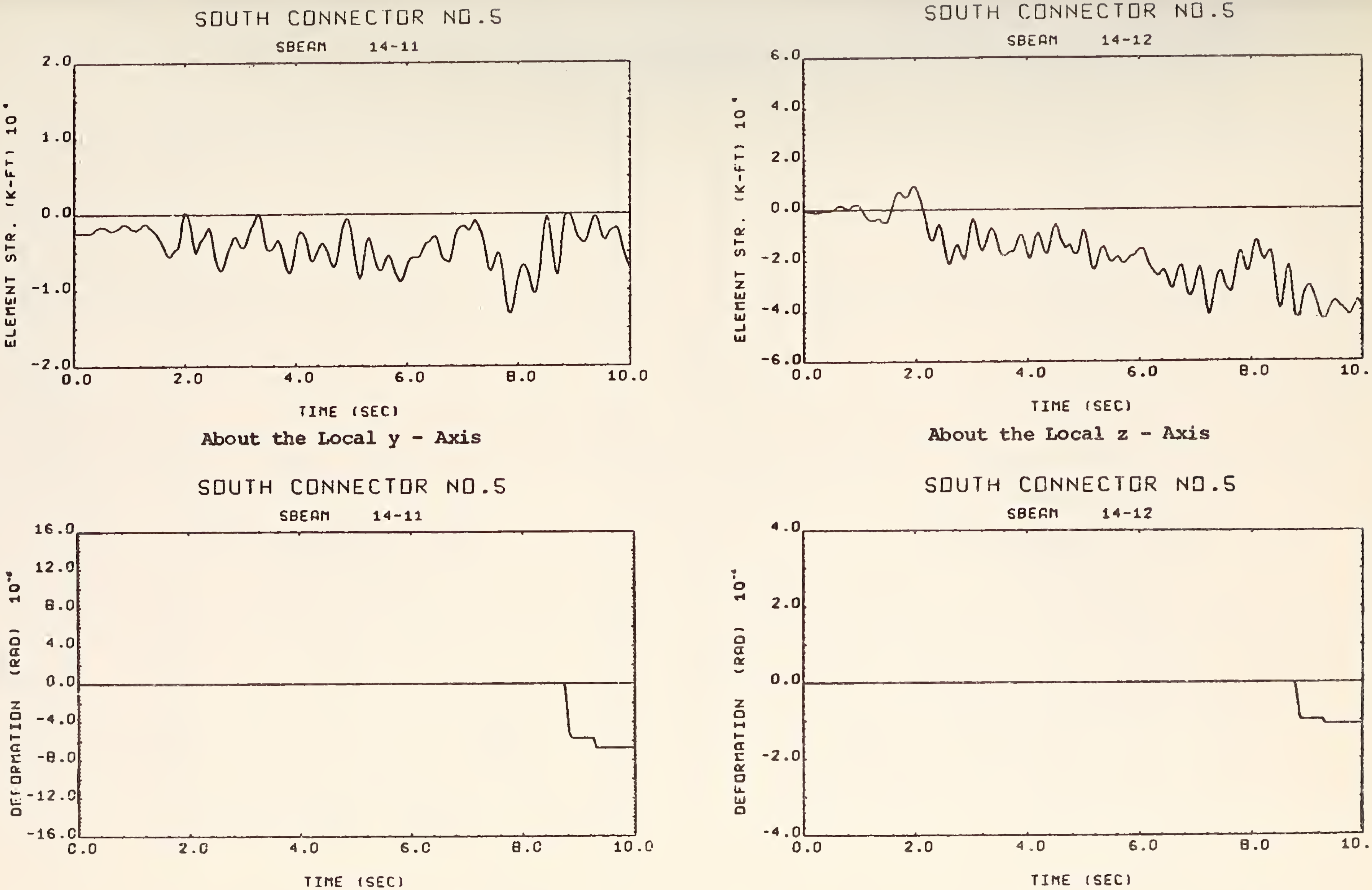


Fig. 60 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

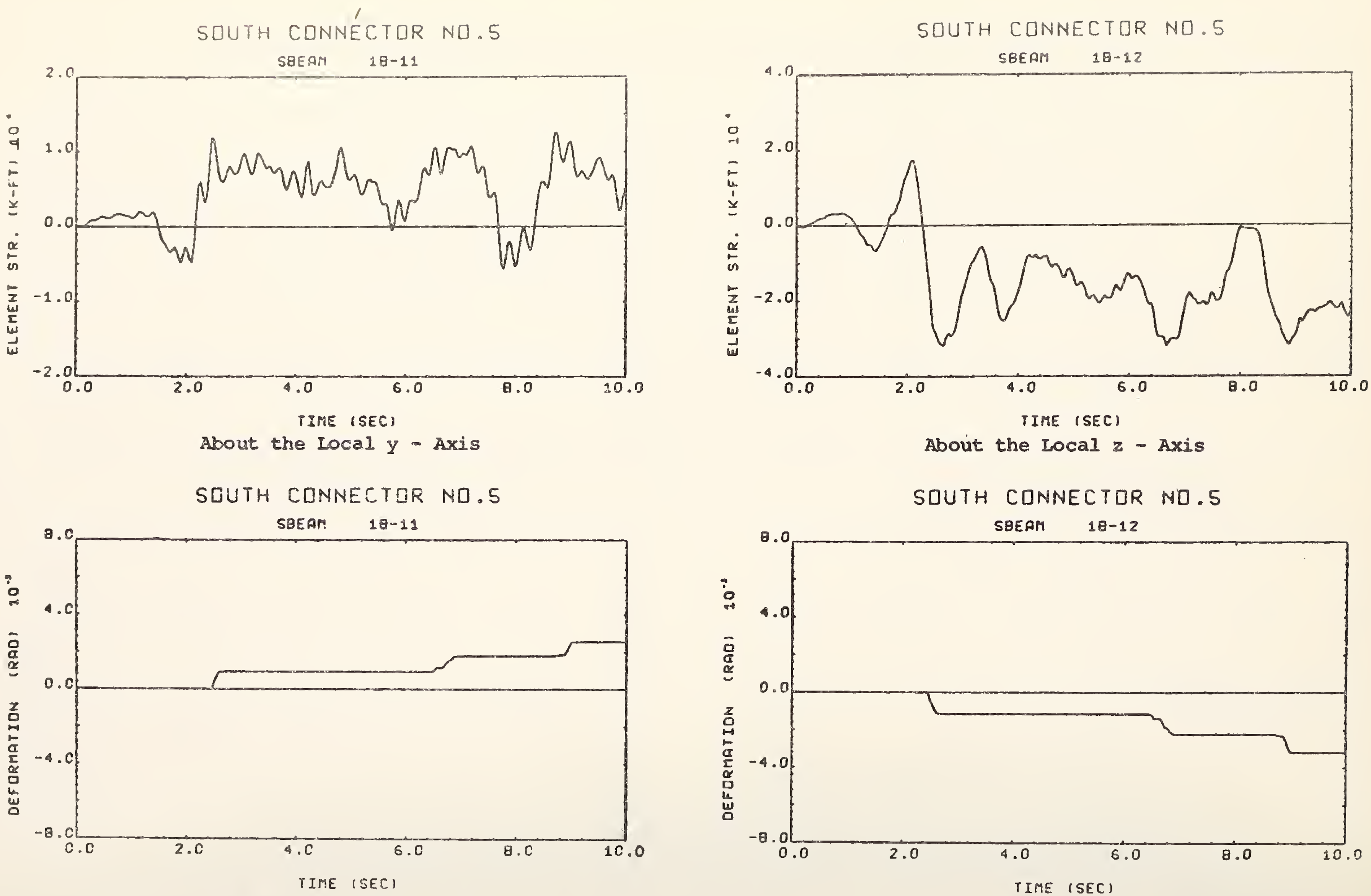
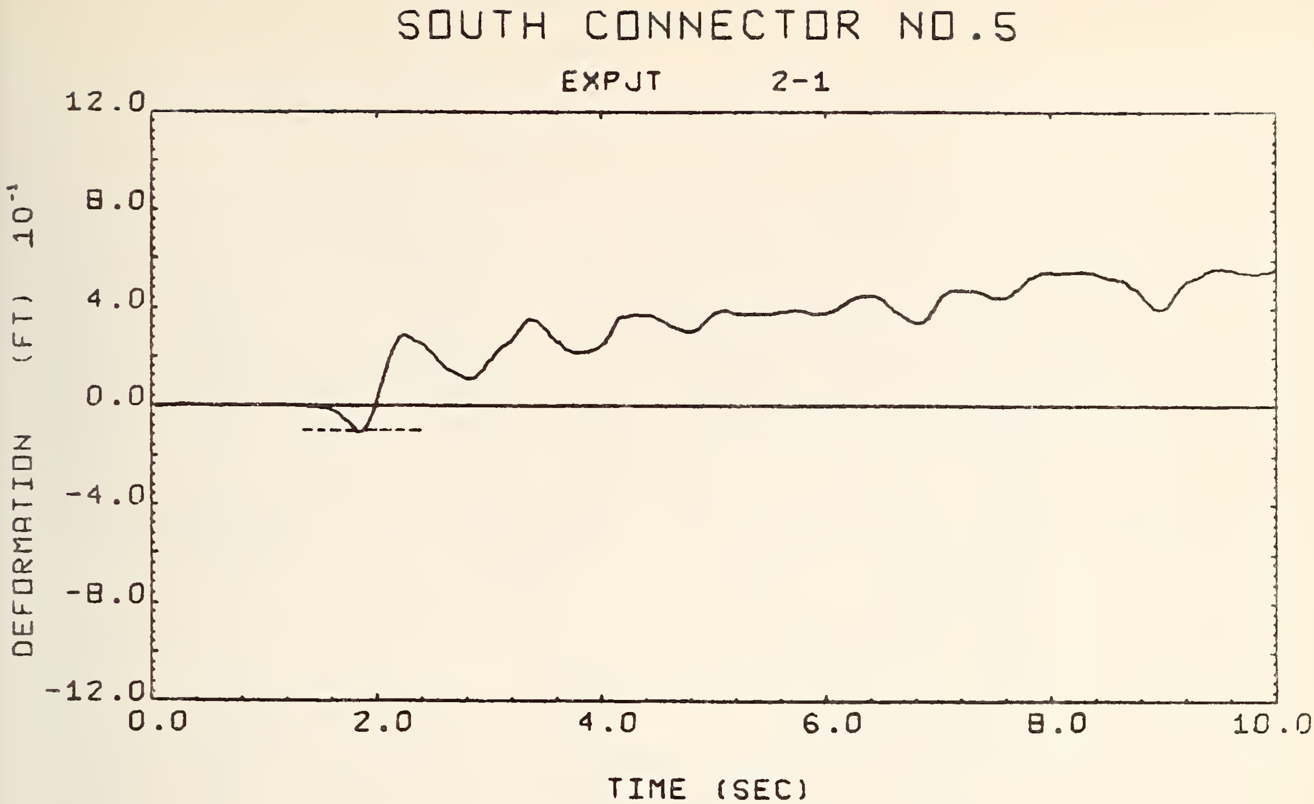
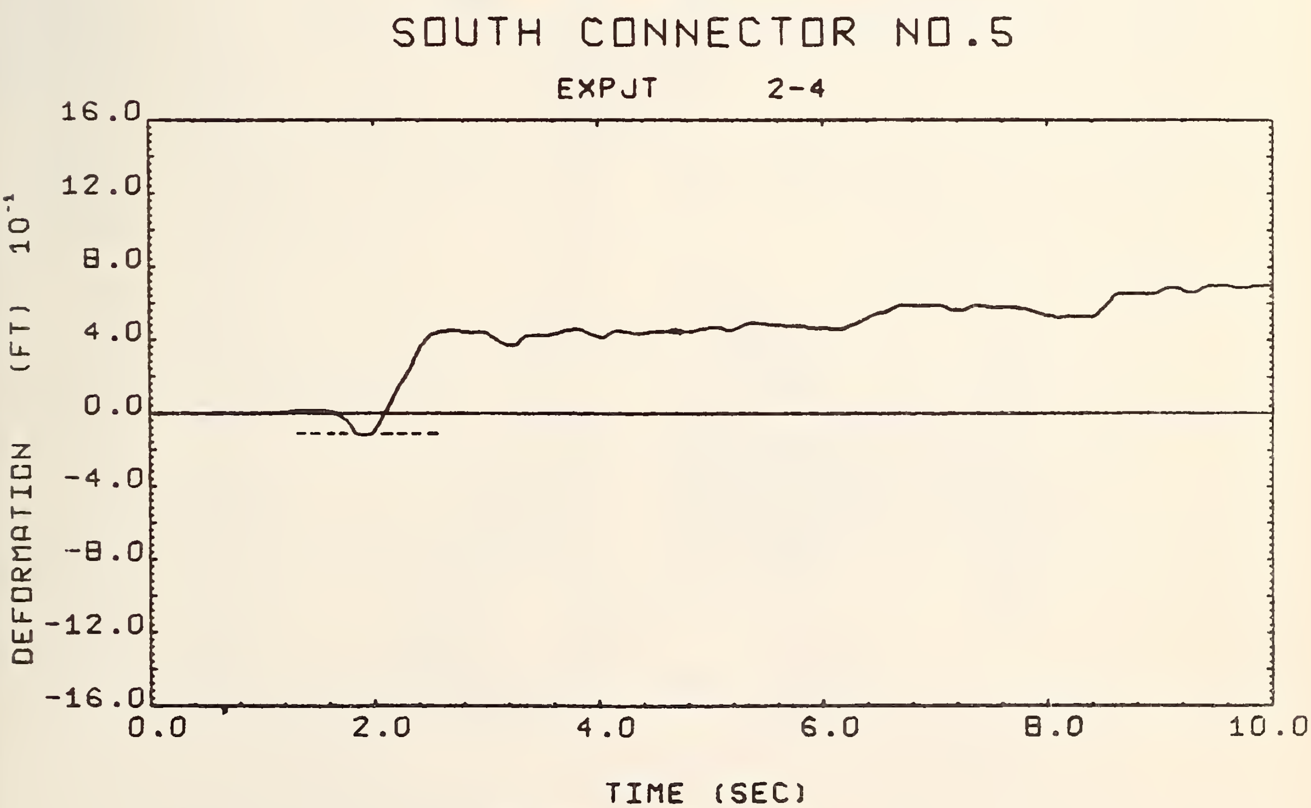


Fig. 61 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 5

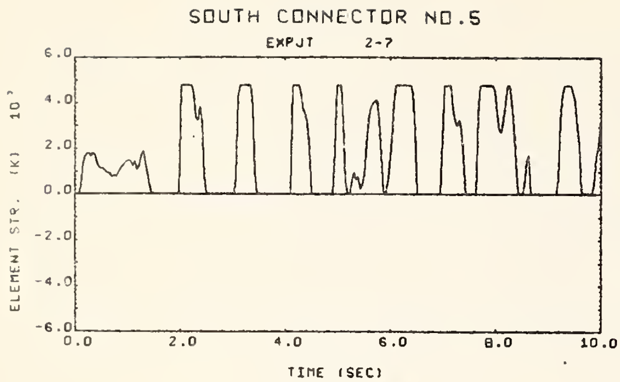


At Inner Edge of the Deck

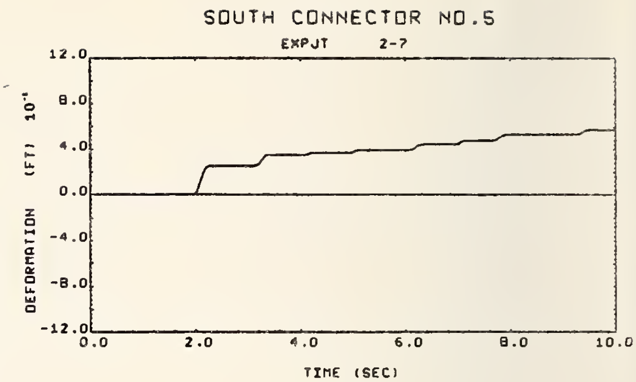


At Outer Edge of the Deck

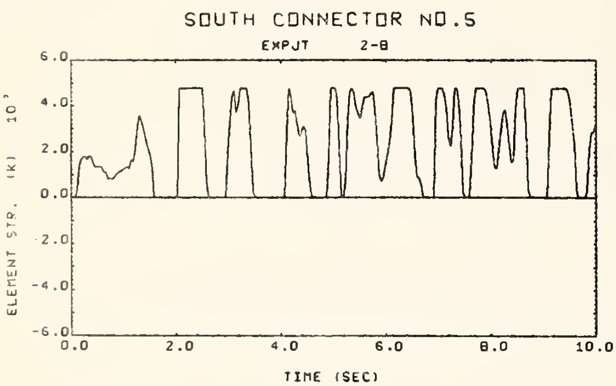
Fig. 62 Longitudinal Joint Separations at Expansion Joint # 2



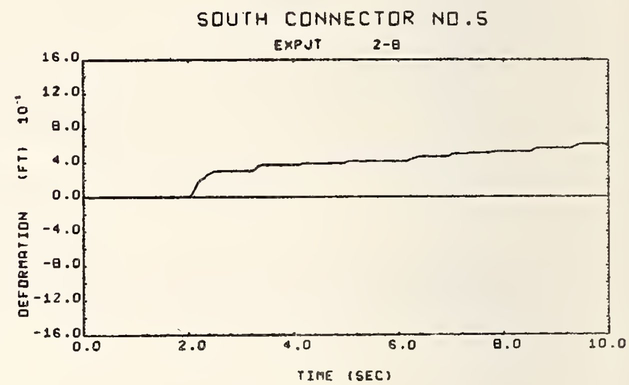
Tie Bar # 1



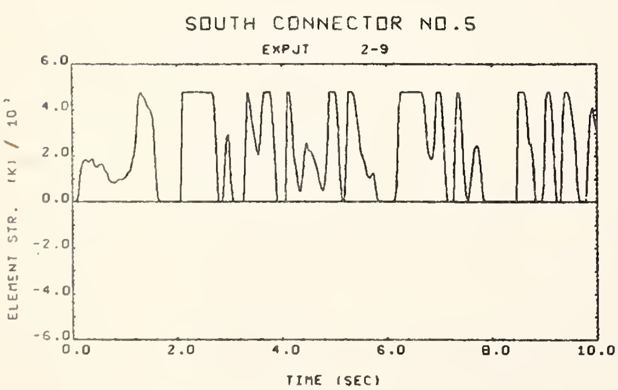
Tie Bar # 1



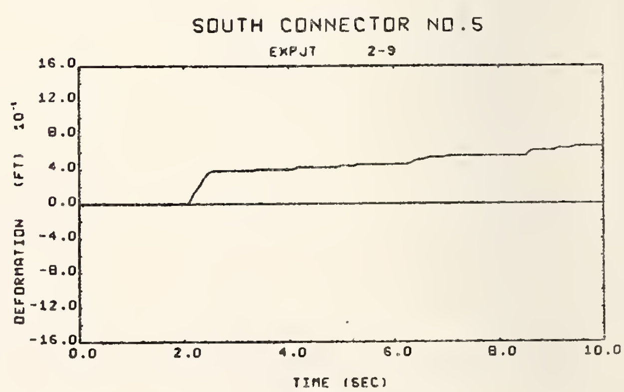
Tie Bar # 2



Tie Bar # 2



Tie Bar # 3



Tie Bar # 3

Fig. 63 Longitudinal Tie Bar Forces and the Corresponding Plastic Elongations at Expansion Joint # 2

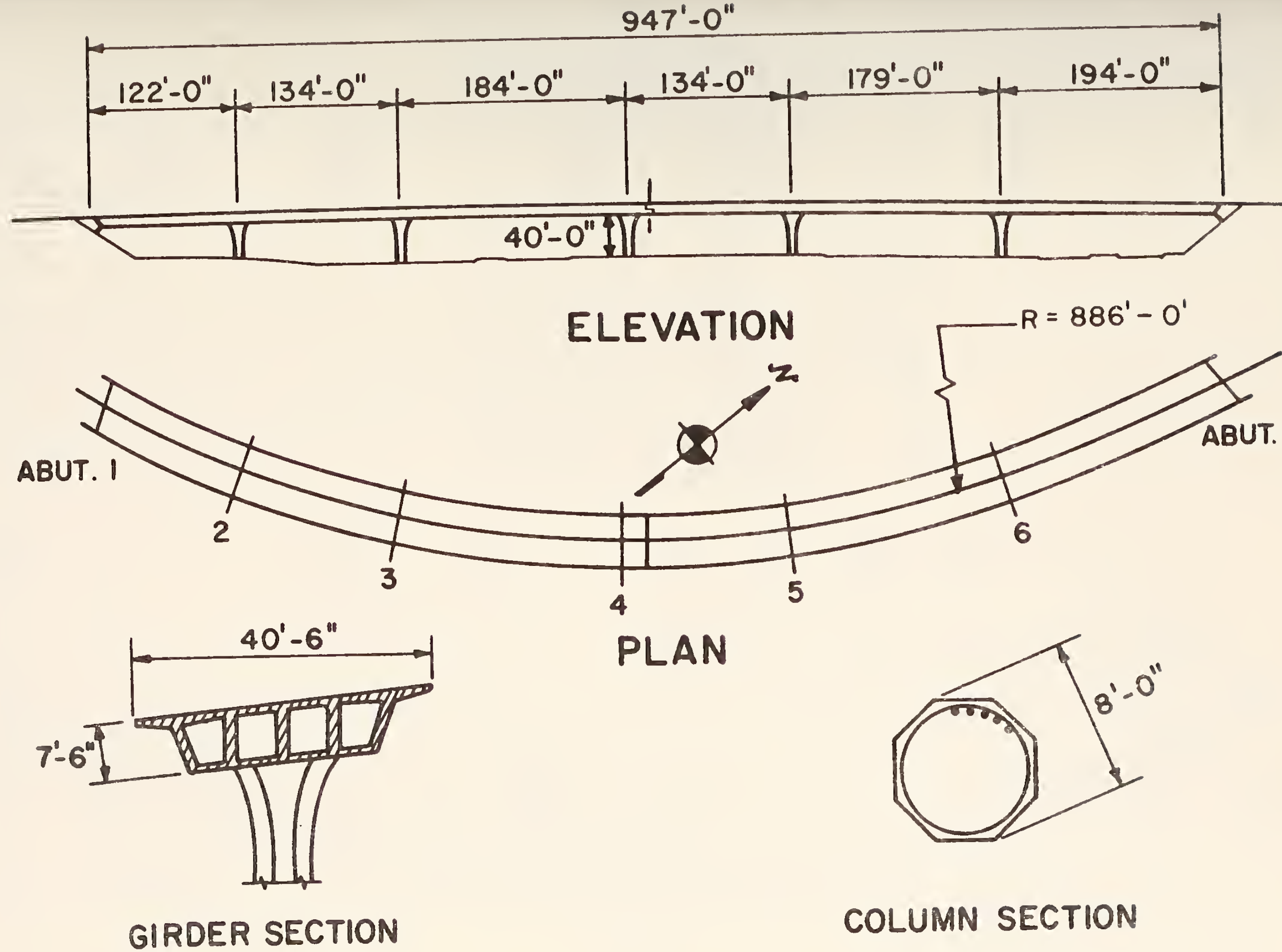
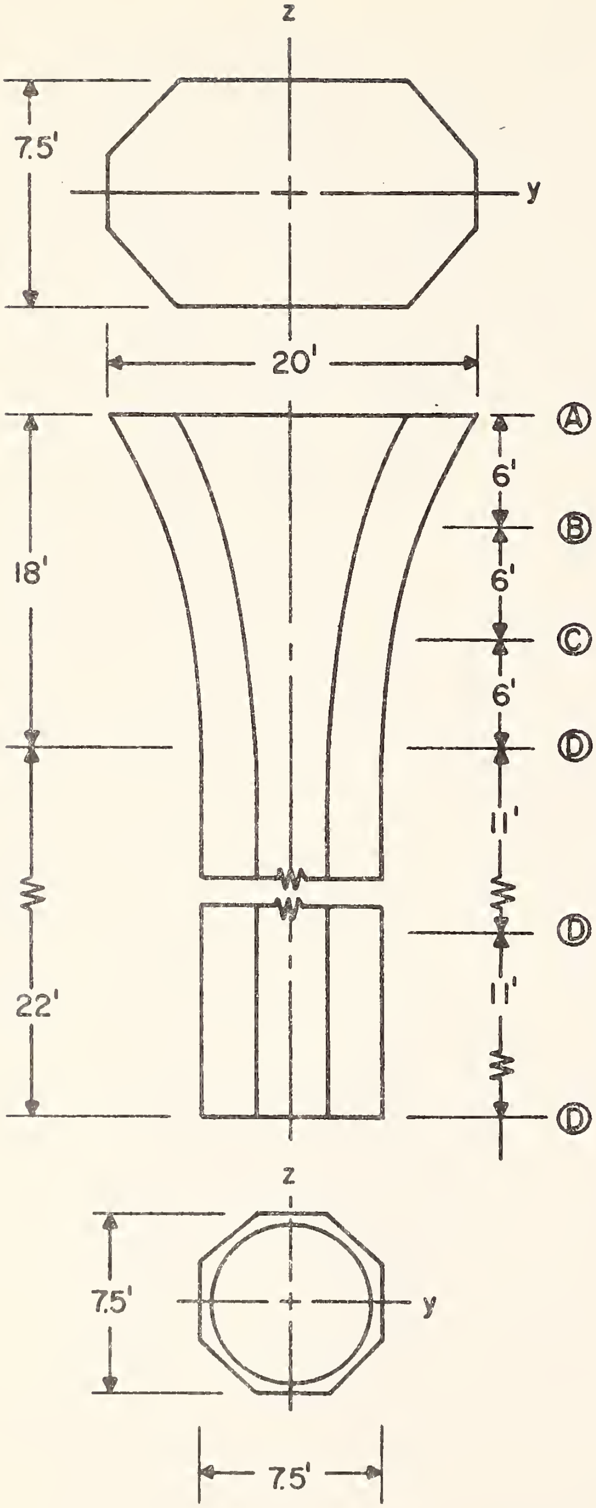


Fig. 64 The Structural System of the Curved Figueroa Street Undercrossing Connector



COLUMN CROSS-SECTION PROPERTIES

CROSS-SECTION	A (ft^2)	I_x (ft^4)	I_y (ft^4)	I_z (ft^4)
A	114.2	1728.	758.	1436.
B	75.1	779.	373.	545.
C	48.3	333.	182.	191.
D	45.6	295.	164.	164.

Fig. 65 Column Cross-Sections of the Curved Figueroa Street Undercrossing Connector

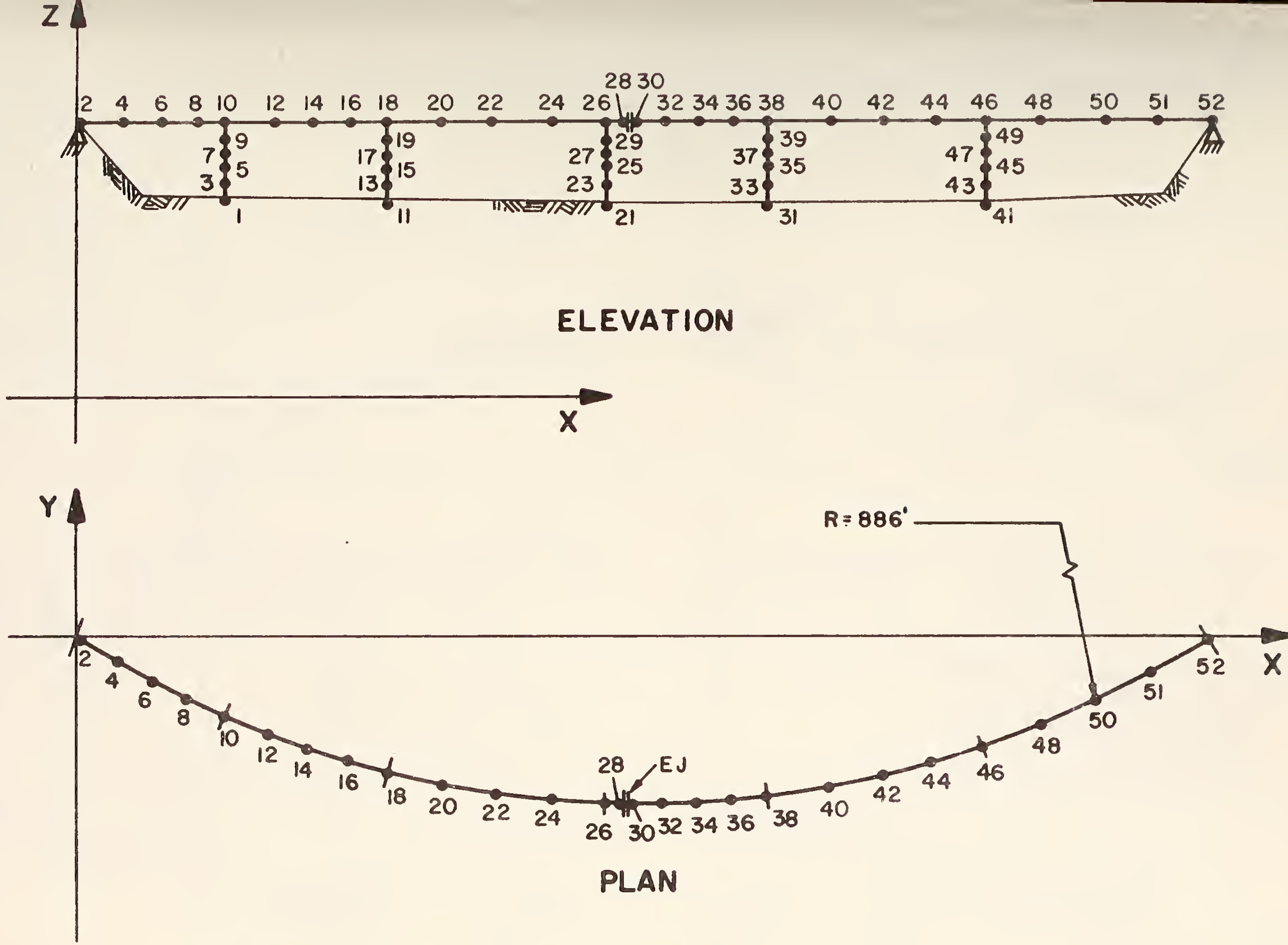


Fig. 66 Lumped Parameter System of the Curved Figueroa Street Undercrossing Connector

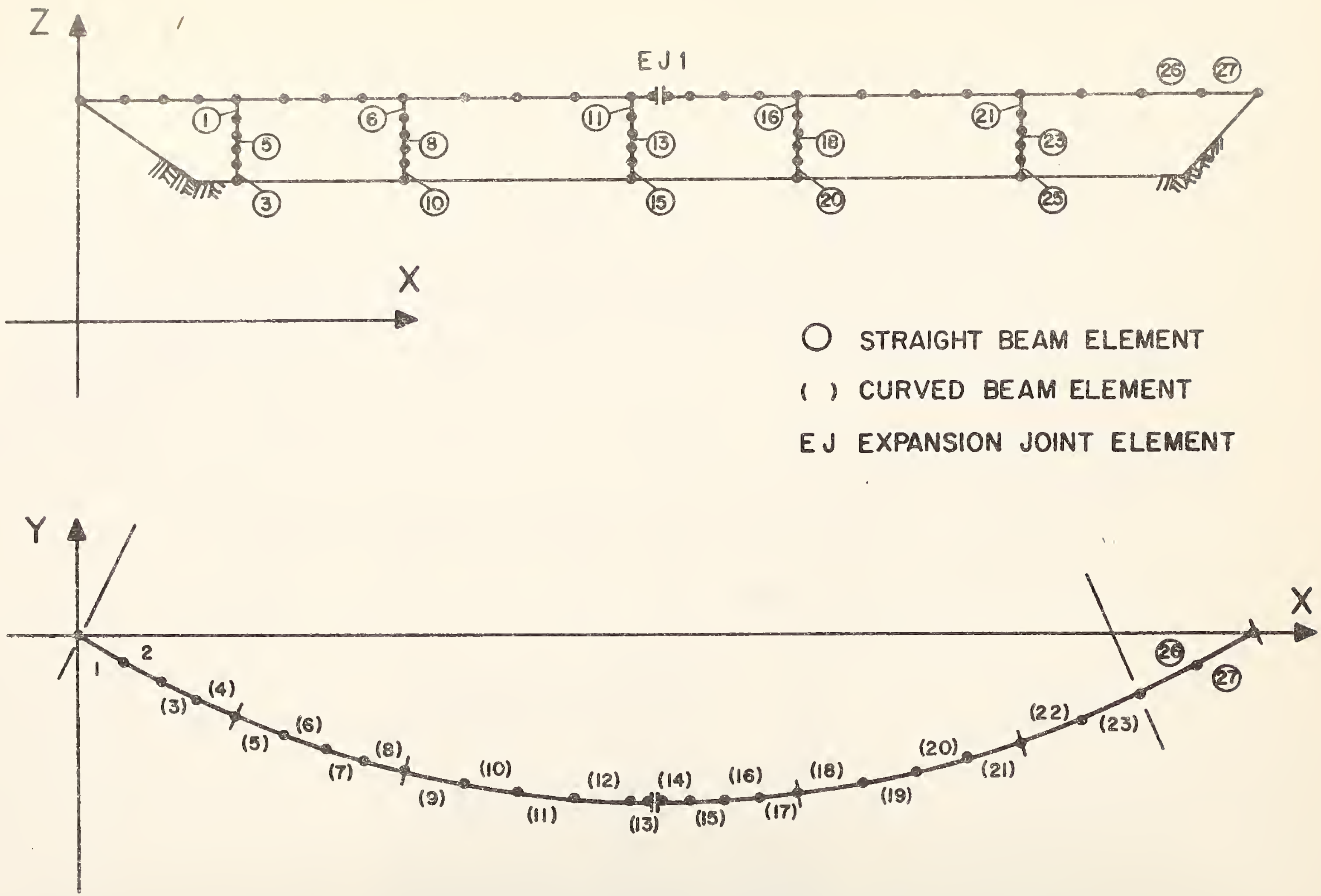


Fig. 67 Finite Element Model of the Curved Figueroa Street U. C. Connector

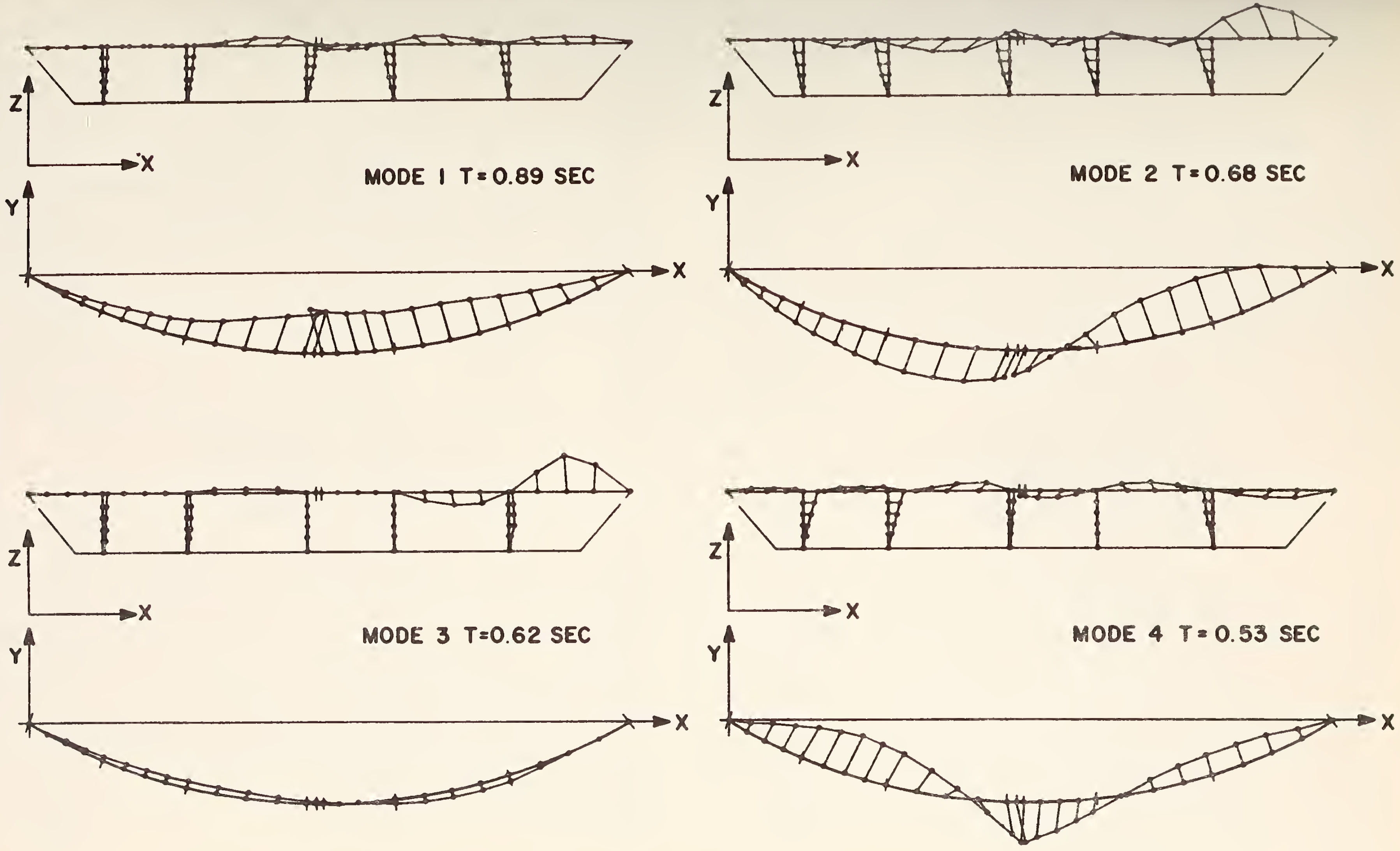


Fig. 68 Mode Shapes of the Curved Figueroa Street Undercrossing Connector: Zero Friction at the Expansion Joint

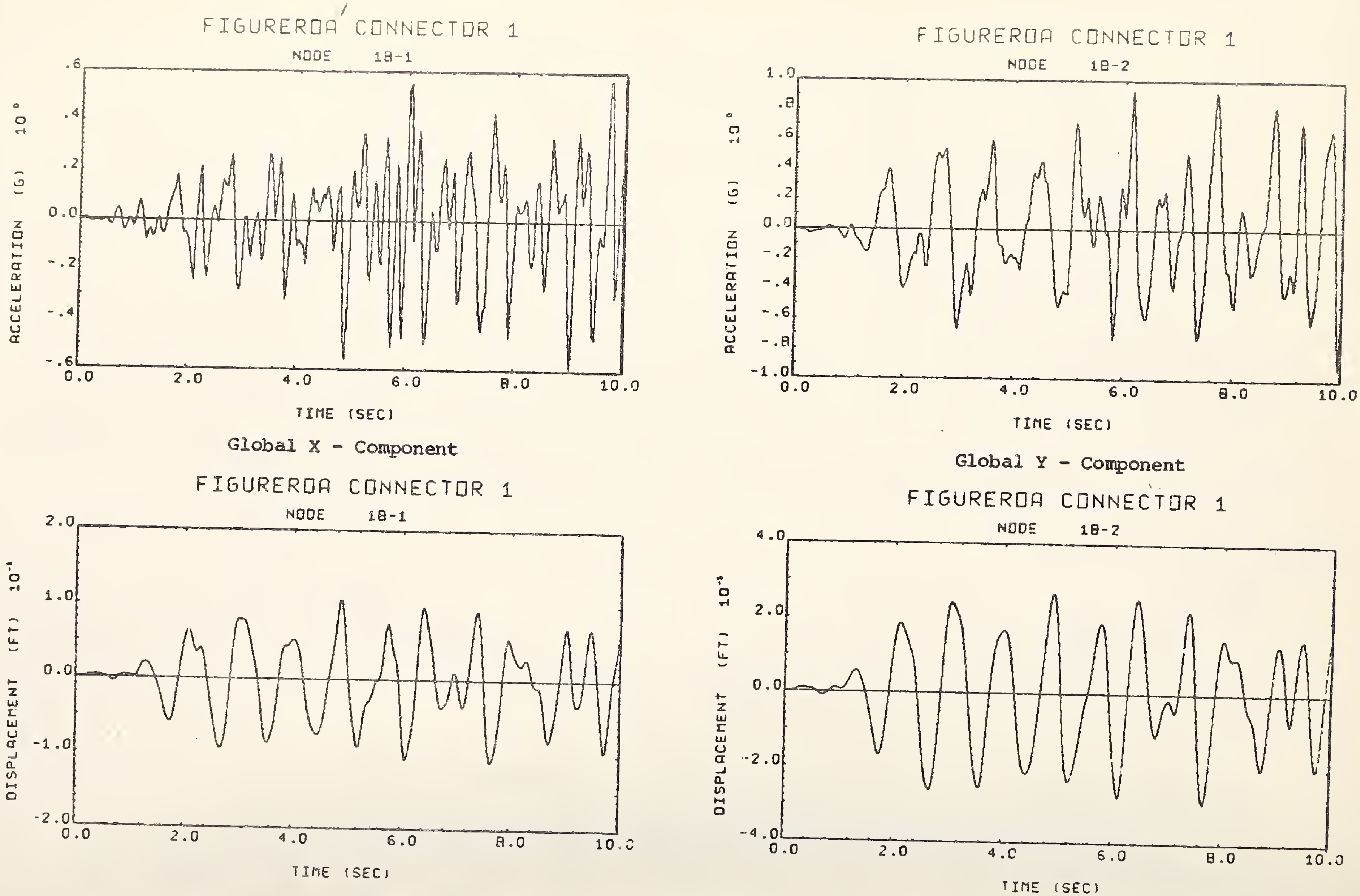


Fig. 69 Horizontal Accelerations and Displacements at the Top of Column # 3

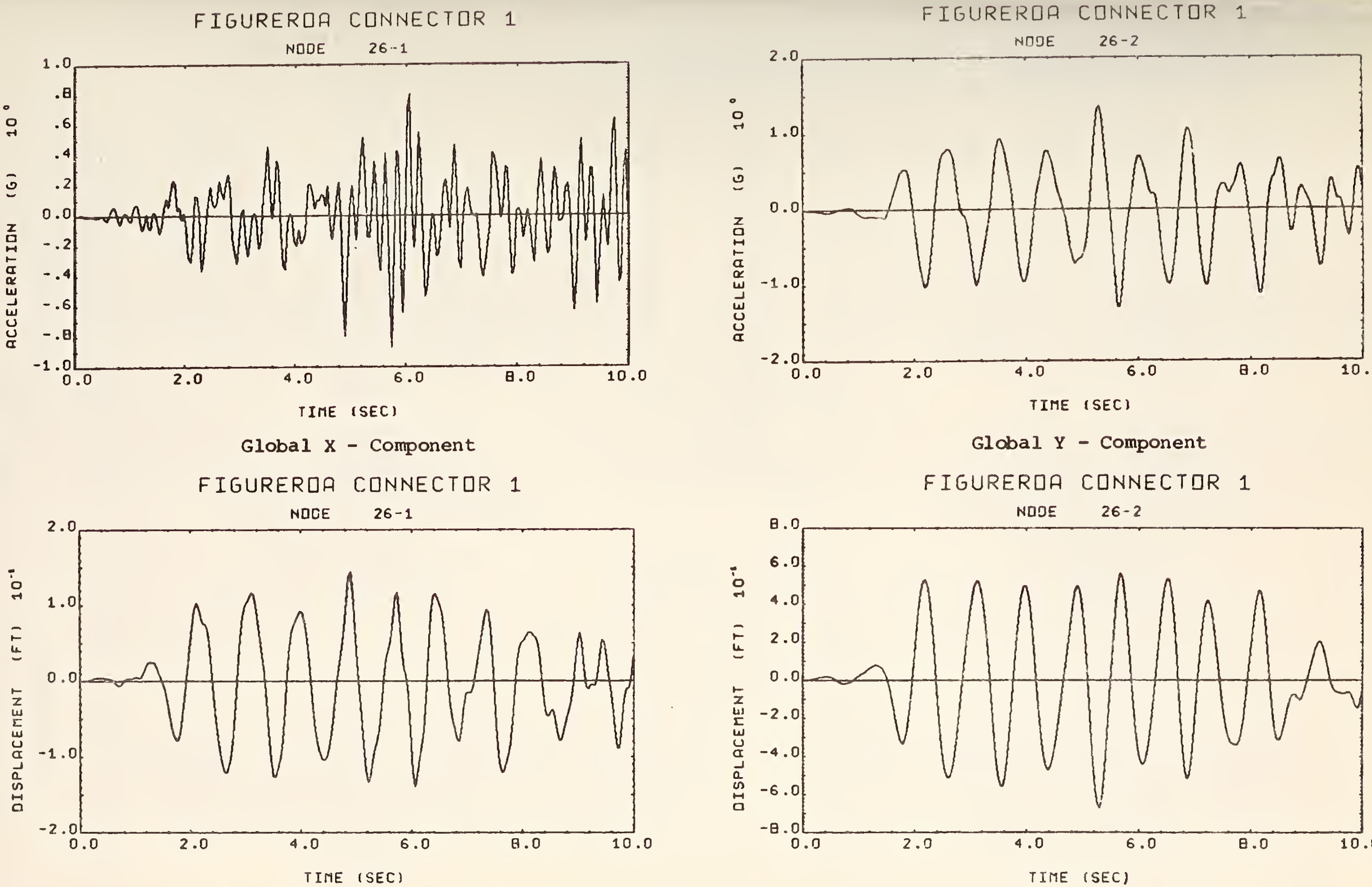


Fig. 70 Horizontal Accelerations and Displacements at the Top of Column # 4

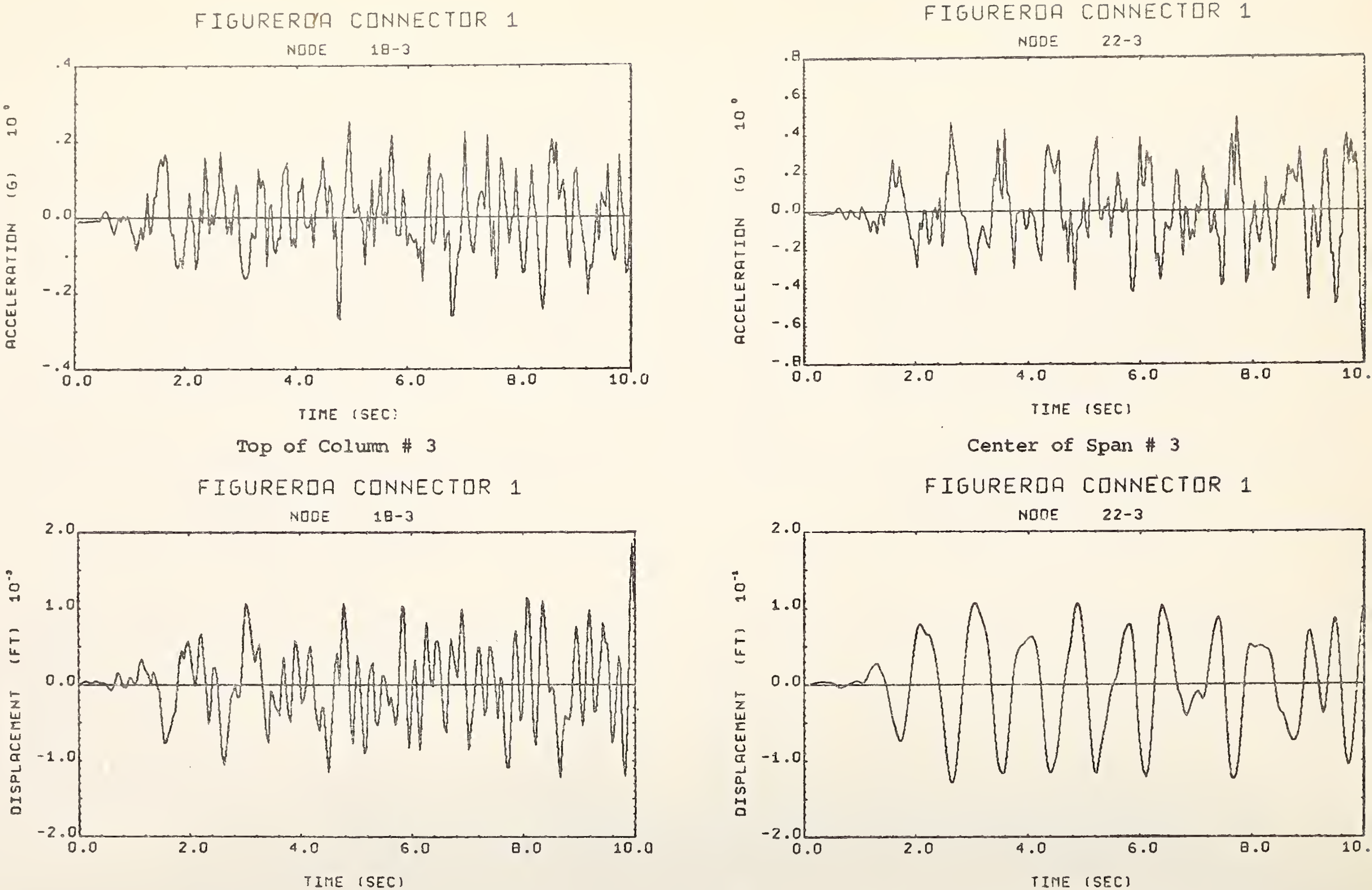
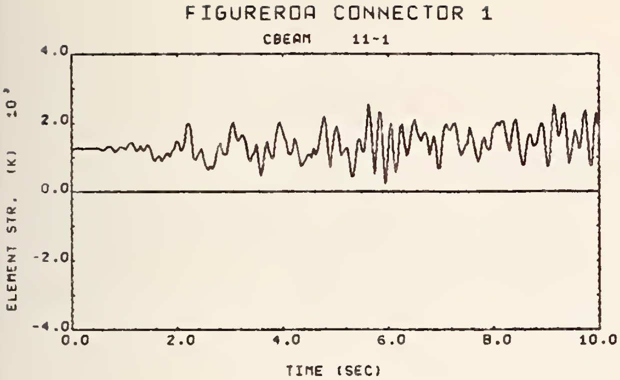
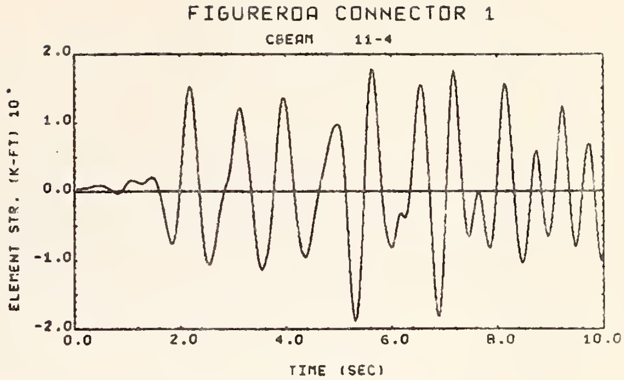


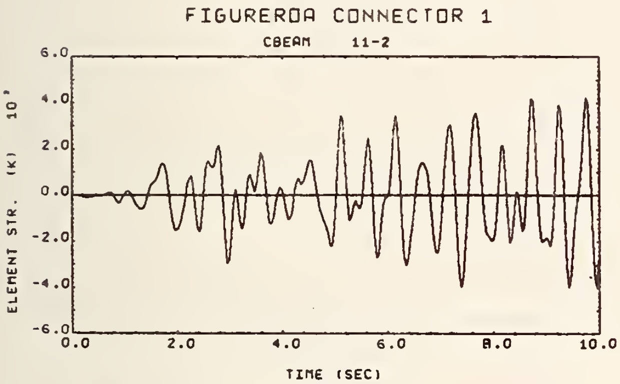
Fig. 71 Vertical Accelerations and Displacements at the Top of Column # 3 and the Center of Span # 3



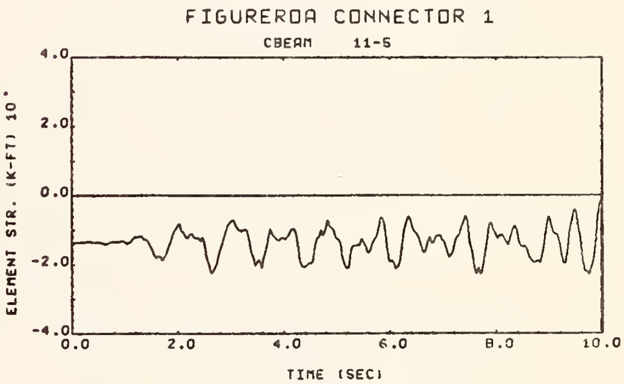
Axial Force



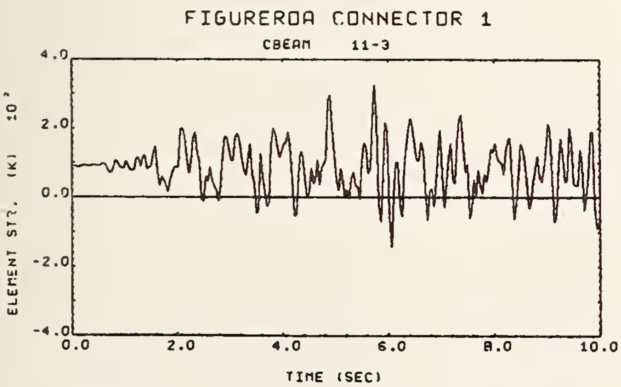
Torsional Moment



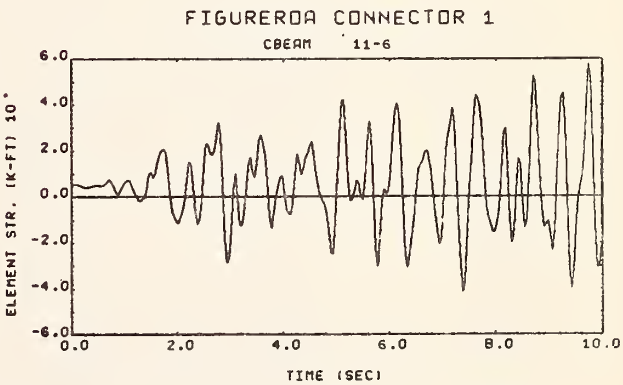
Shear in the Local y - Direction



Moment about Local y - Axis



Shear in the Local z - Direction



Moment about Local z - Axis

Fig. 72 Generalized Forces in the Girder at the Center of Span # 3

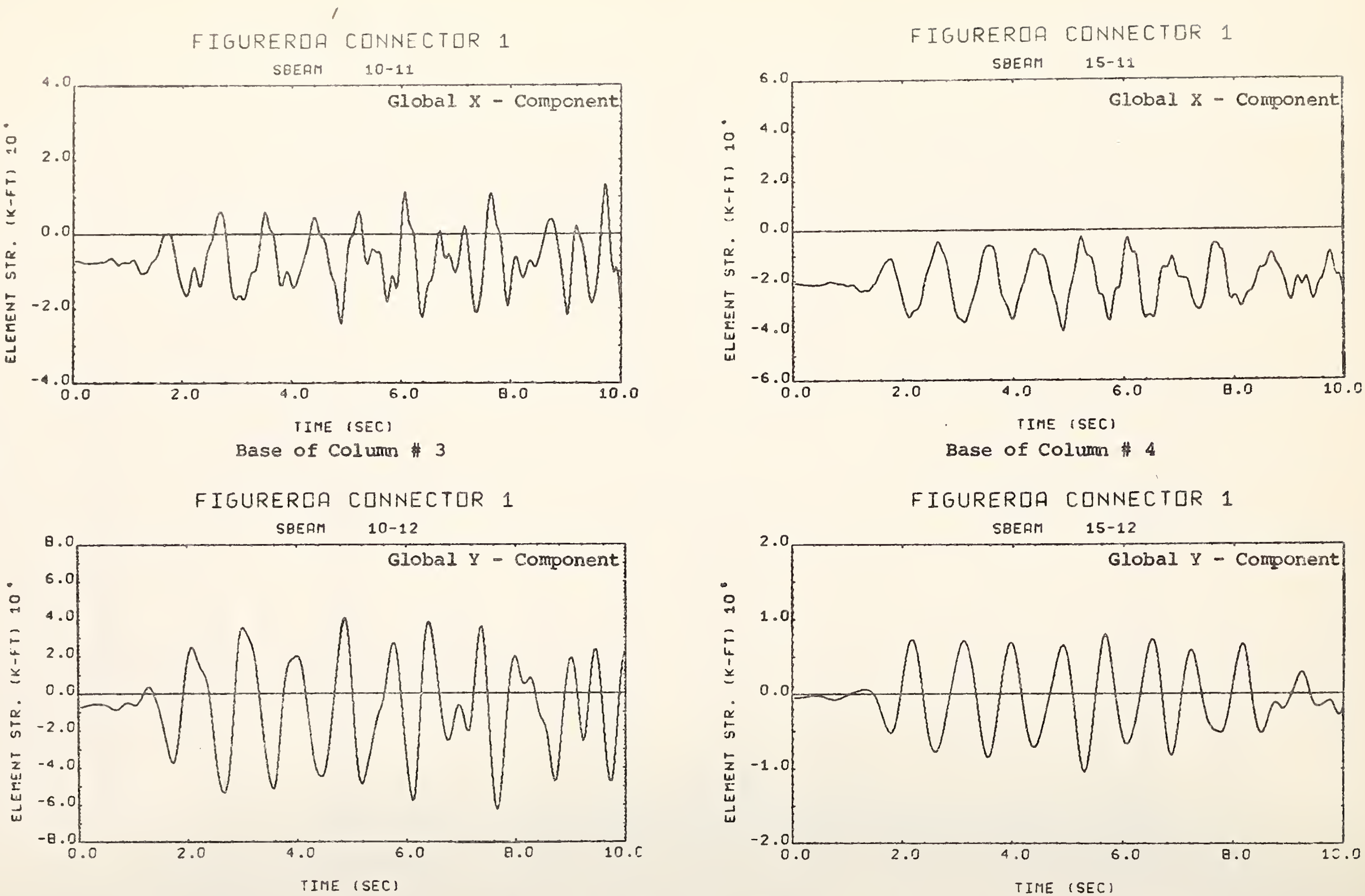
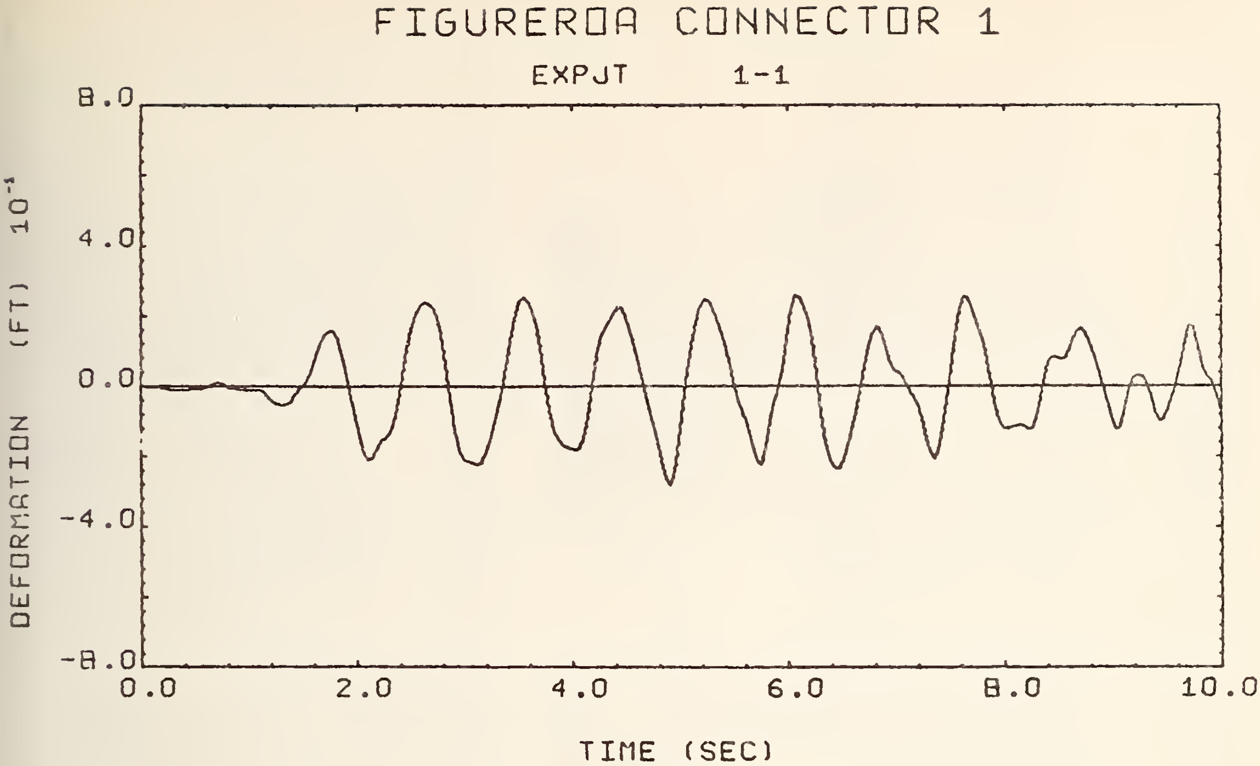
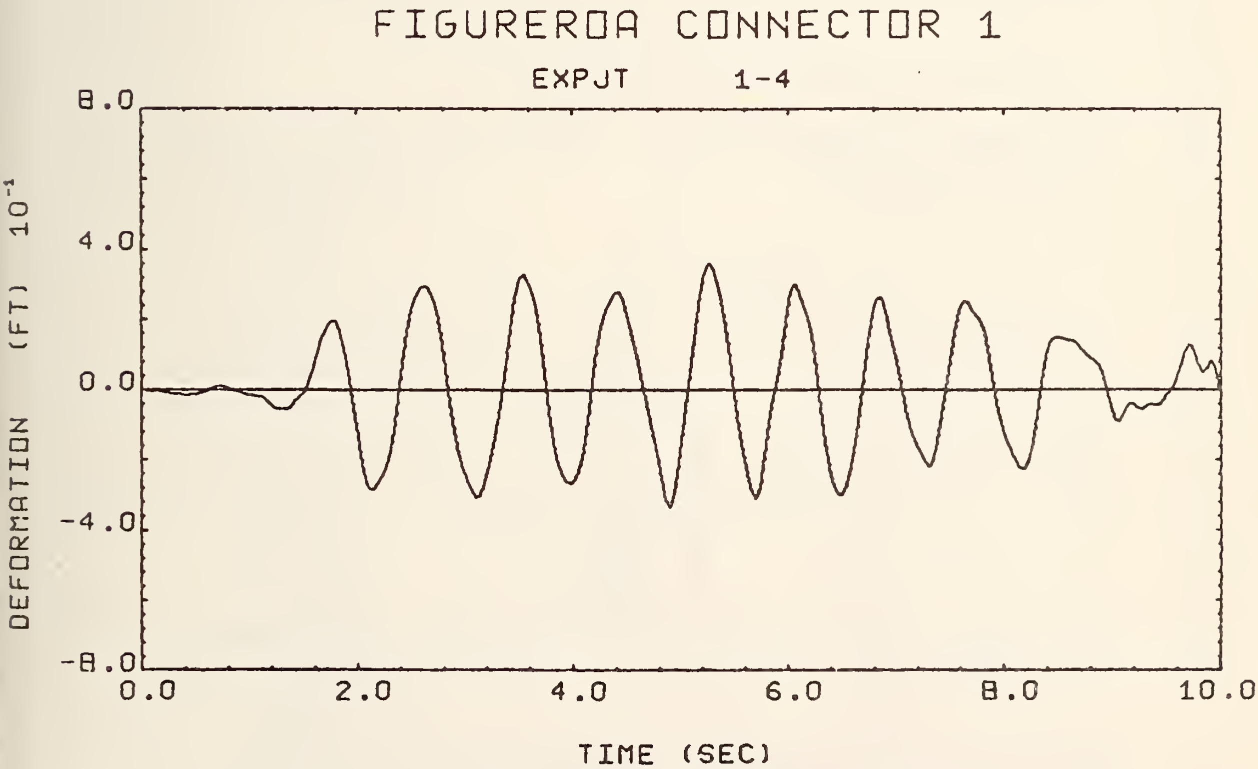


Fig. 73 Bending Moments at the Base of Columns # 3 and # 4



At Inner Edge of the Deck



At Outer Edge of the Deck

Fig. 74 Longitudinal Joint Separations at Expansion Joint # 1

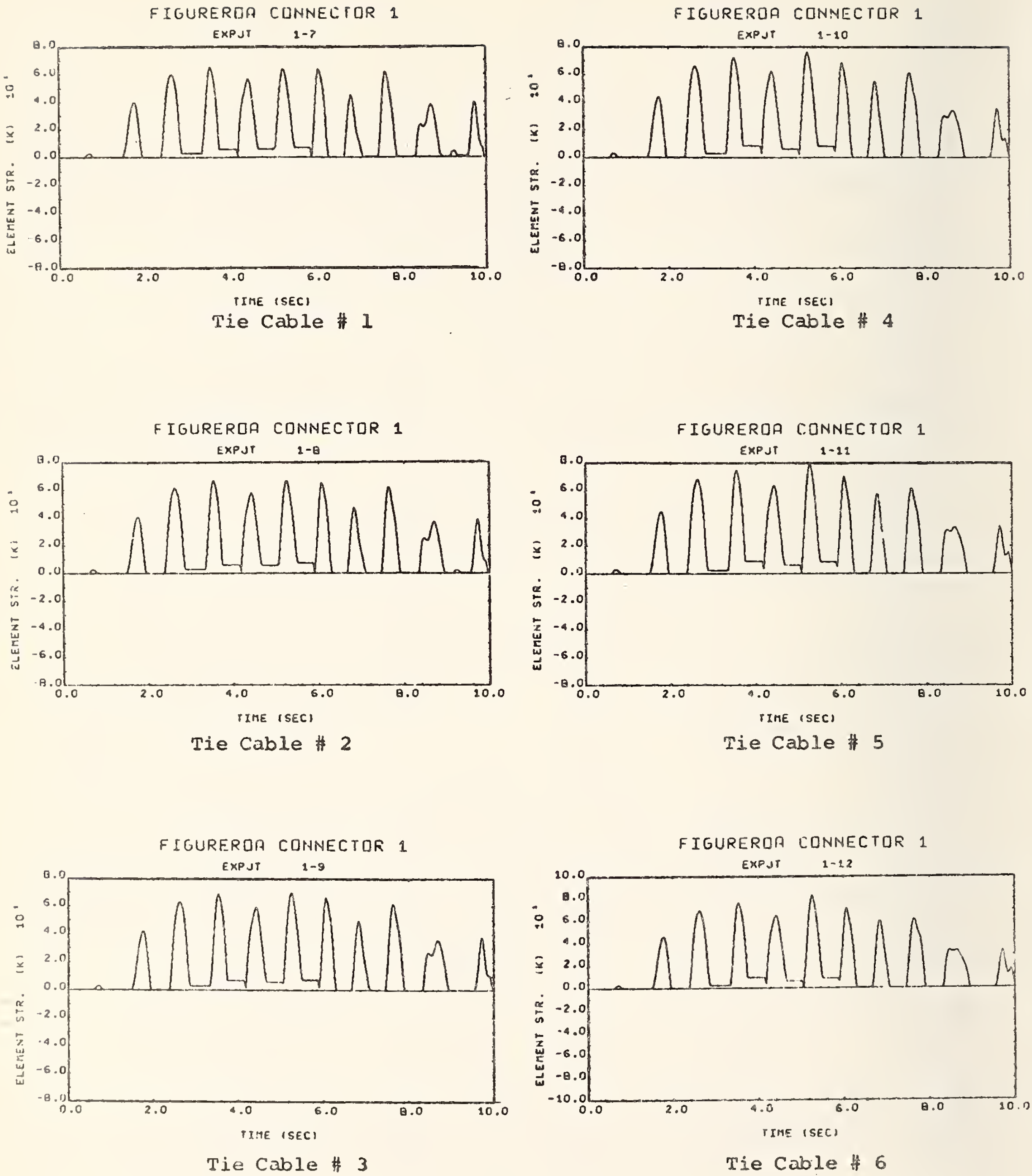


Fig. 75 Longitudinal Tie Cable Forces at Expansion Joint # 1

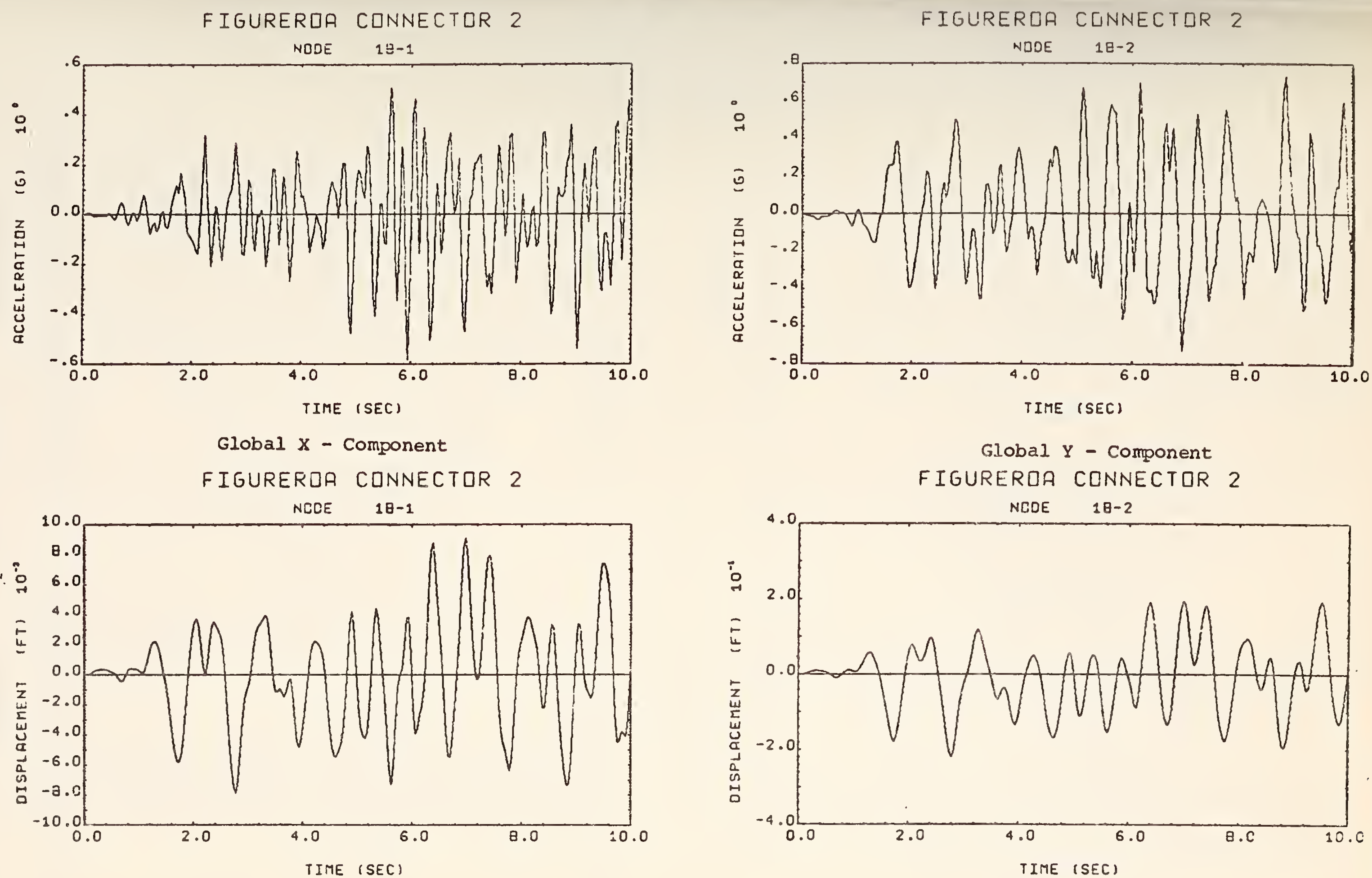


Fig. 76 Horizontal Accelerations and Displacements at the Top of Column # 3

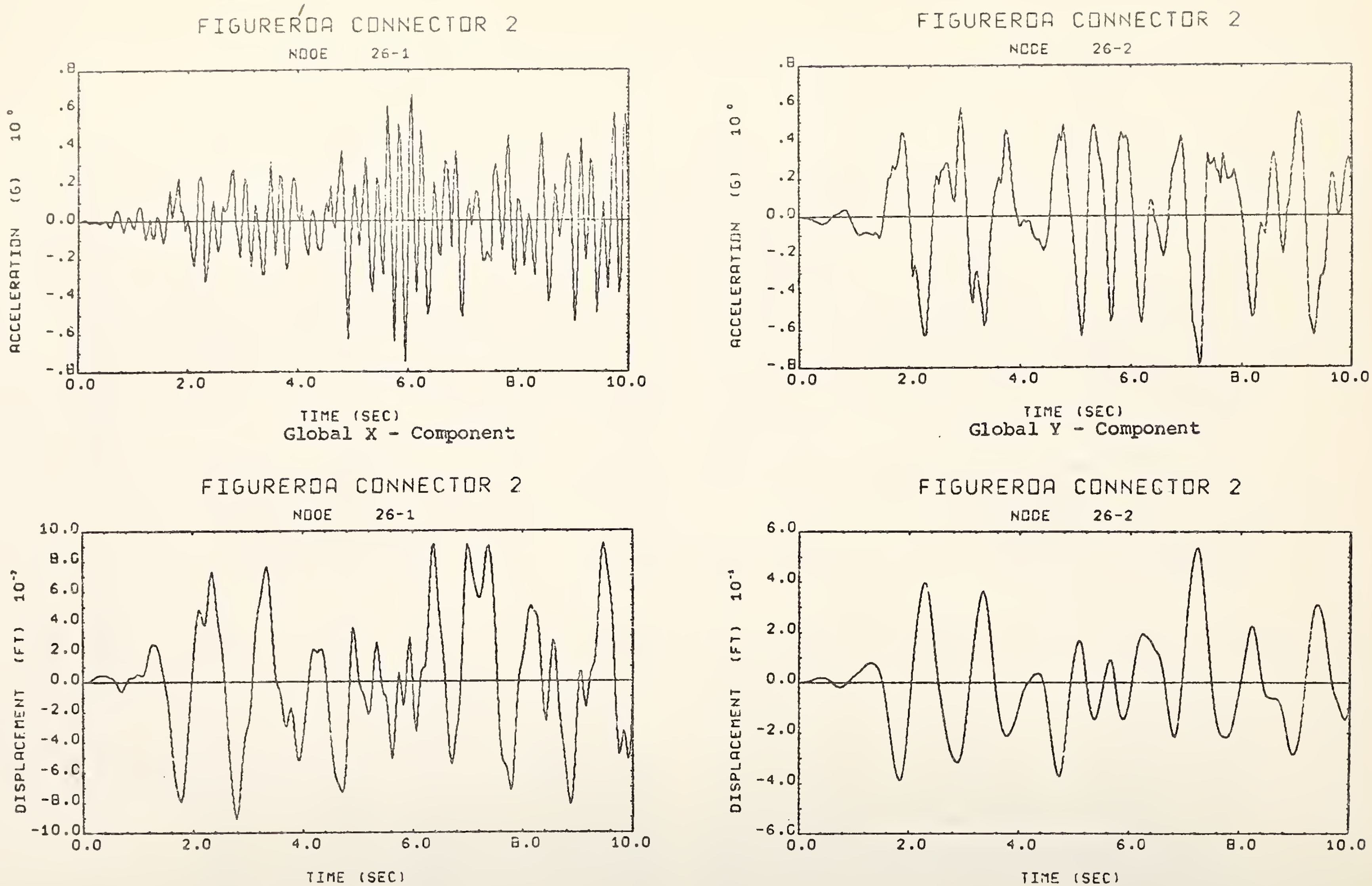
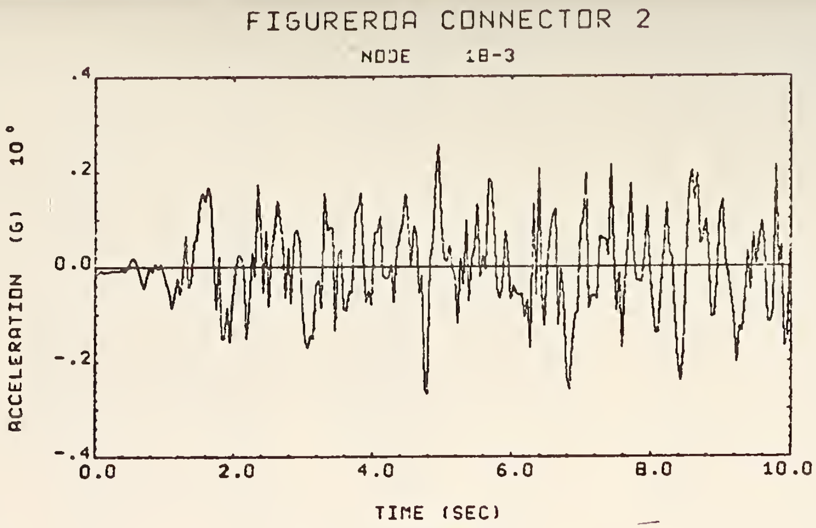
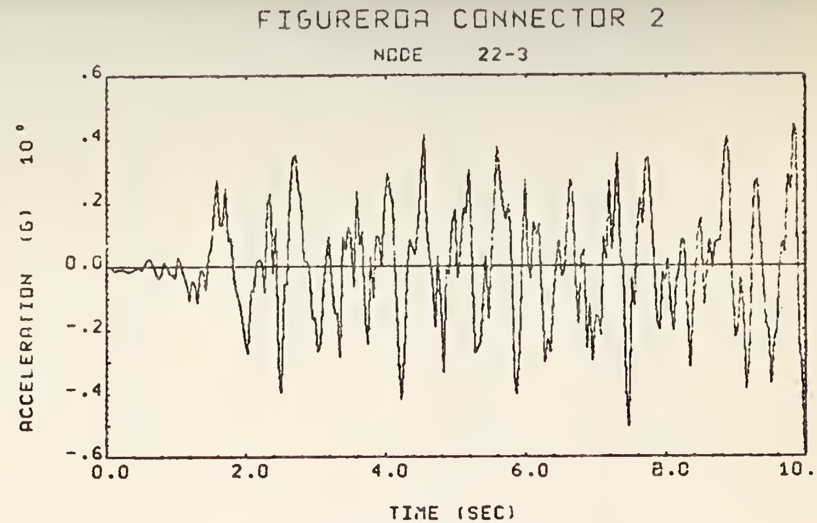


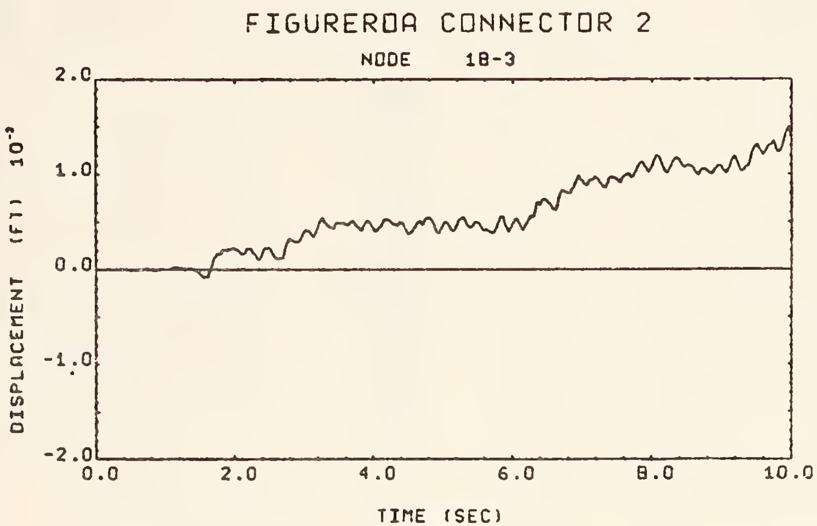
Fig. 77 Horizontal Accelerations and Displacements at the Top of Column # 4



Top of Column # 3



Center of Span # 3



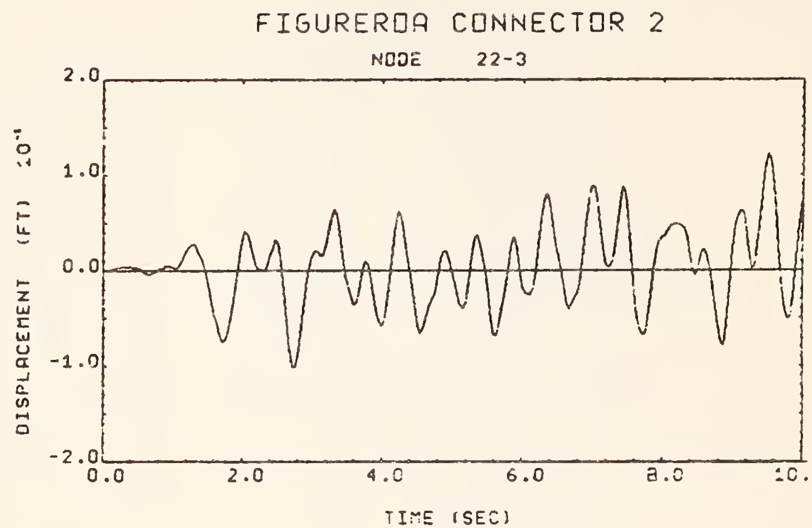
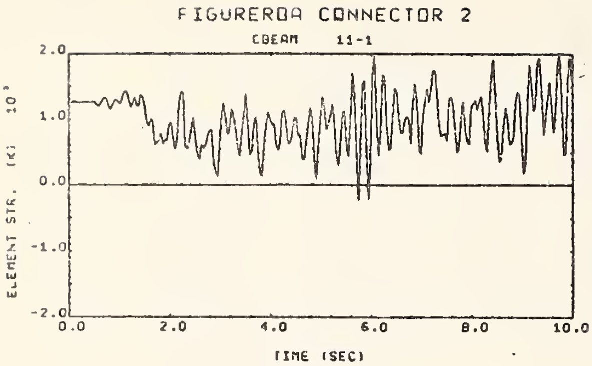
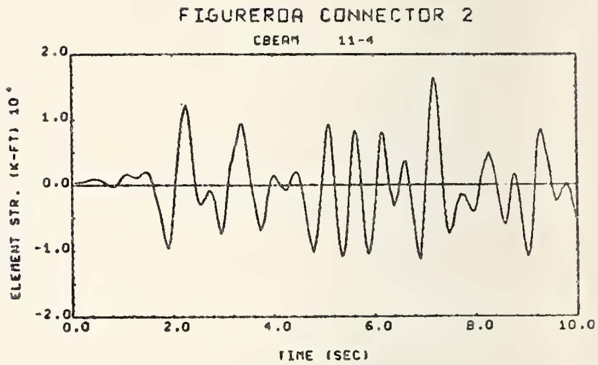


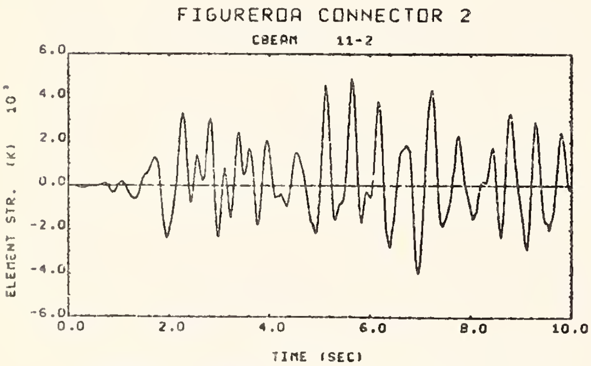
Fig. 78 Vertical Accelerations and Displacements at the Top of Column # 3 and the Center of Span # 3



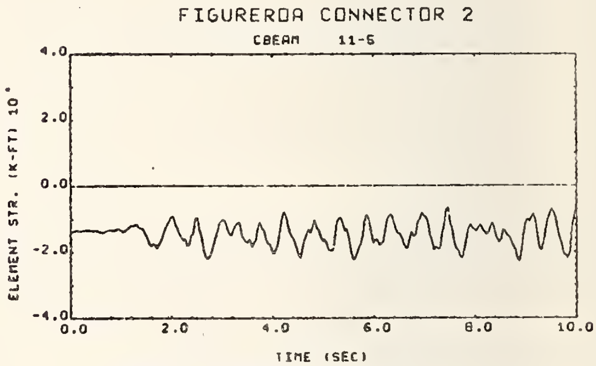
Axial Force



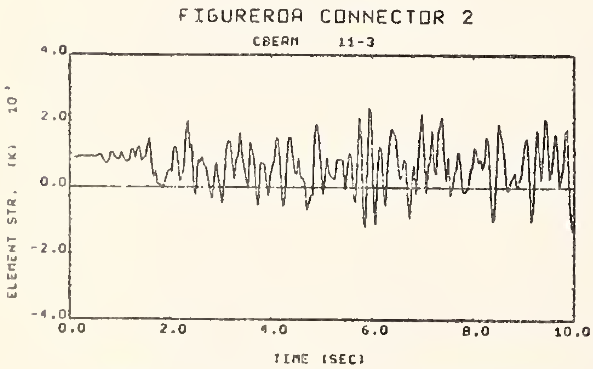
Torsional Moment



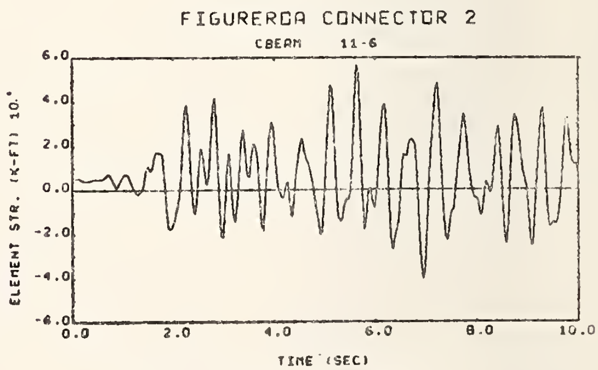
Shear in the Local y - Direction



Moment about the Local y - Axis



Shear in the Local z - Direction



Moment about the Local z - Axis

Fig. 79 Generalized Forces in the Girder at the Center of Span # 3

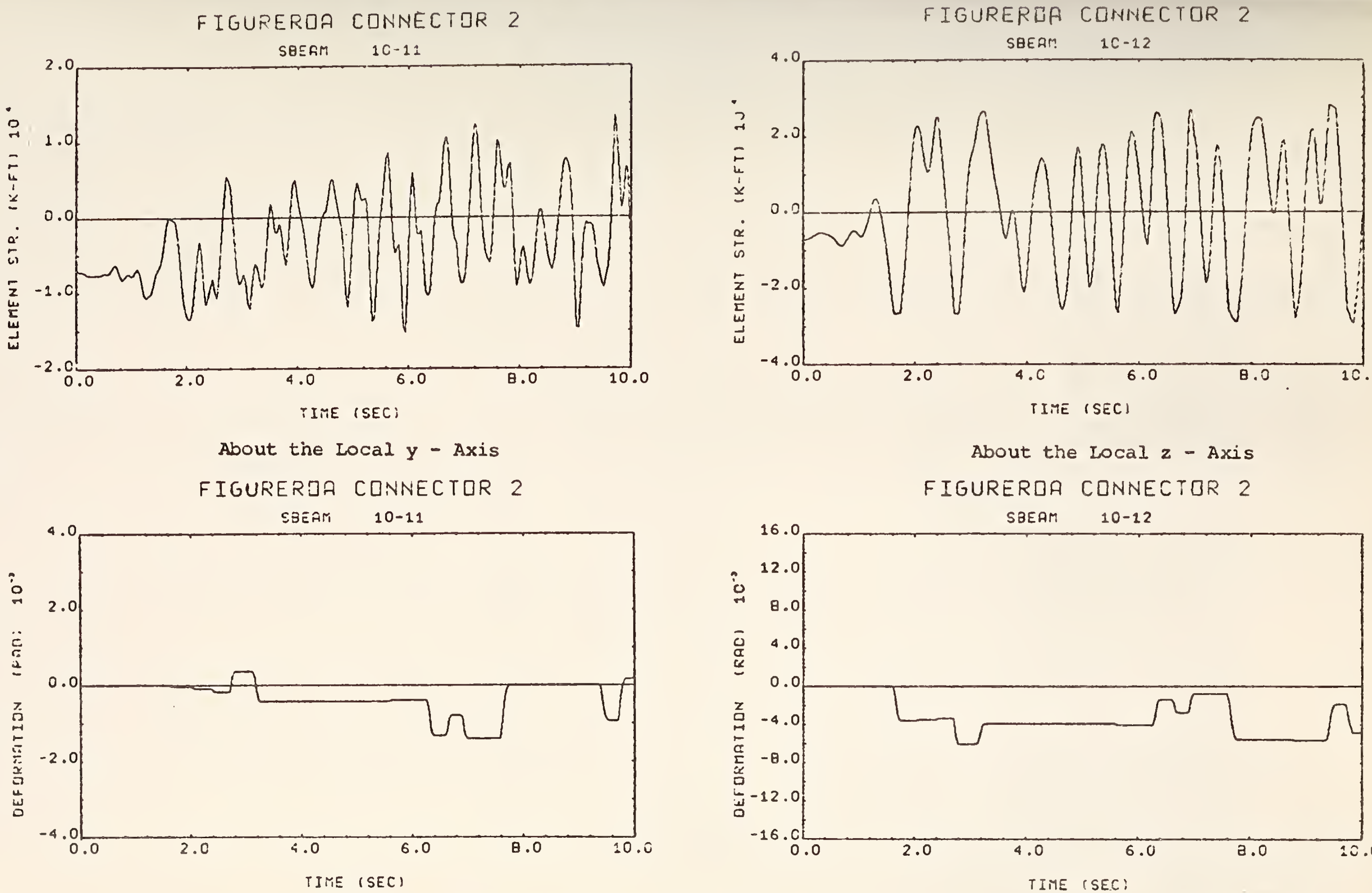
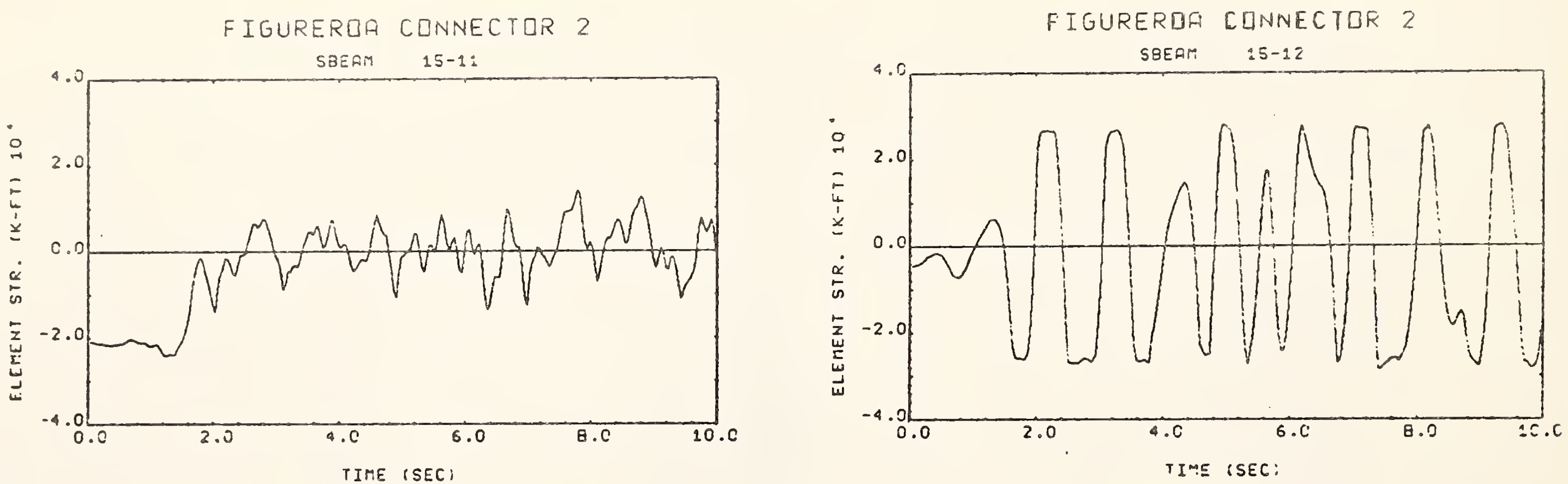


Fig. 80 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3



About the Local y - Axis

About the Local z - Axis

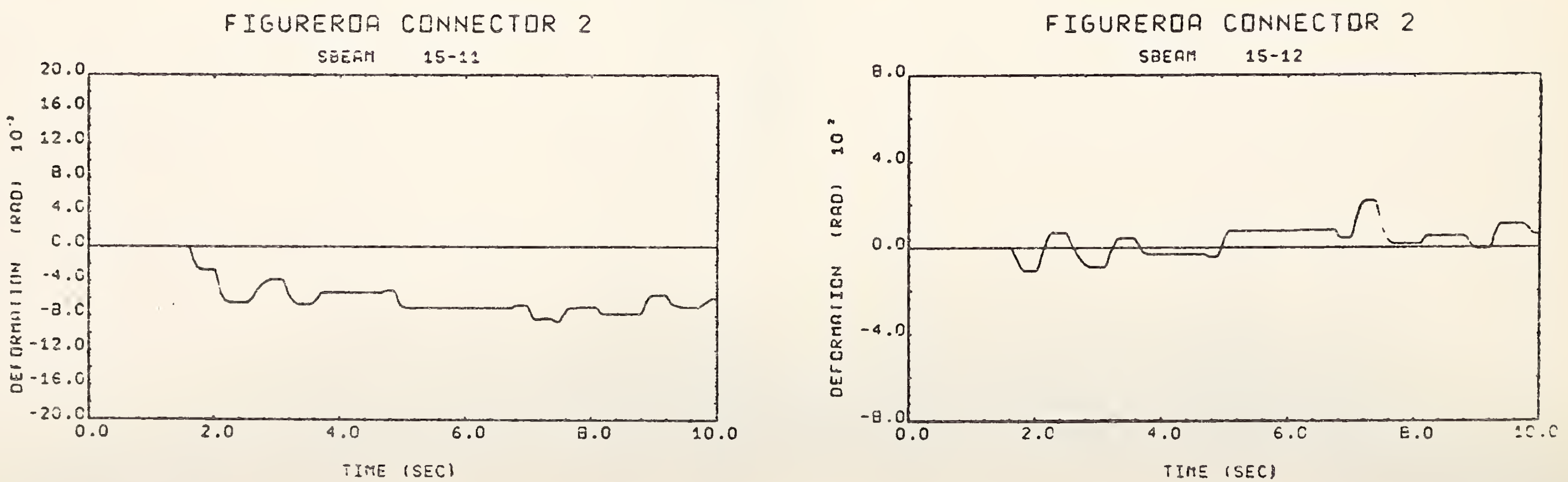
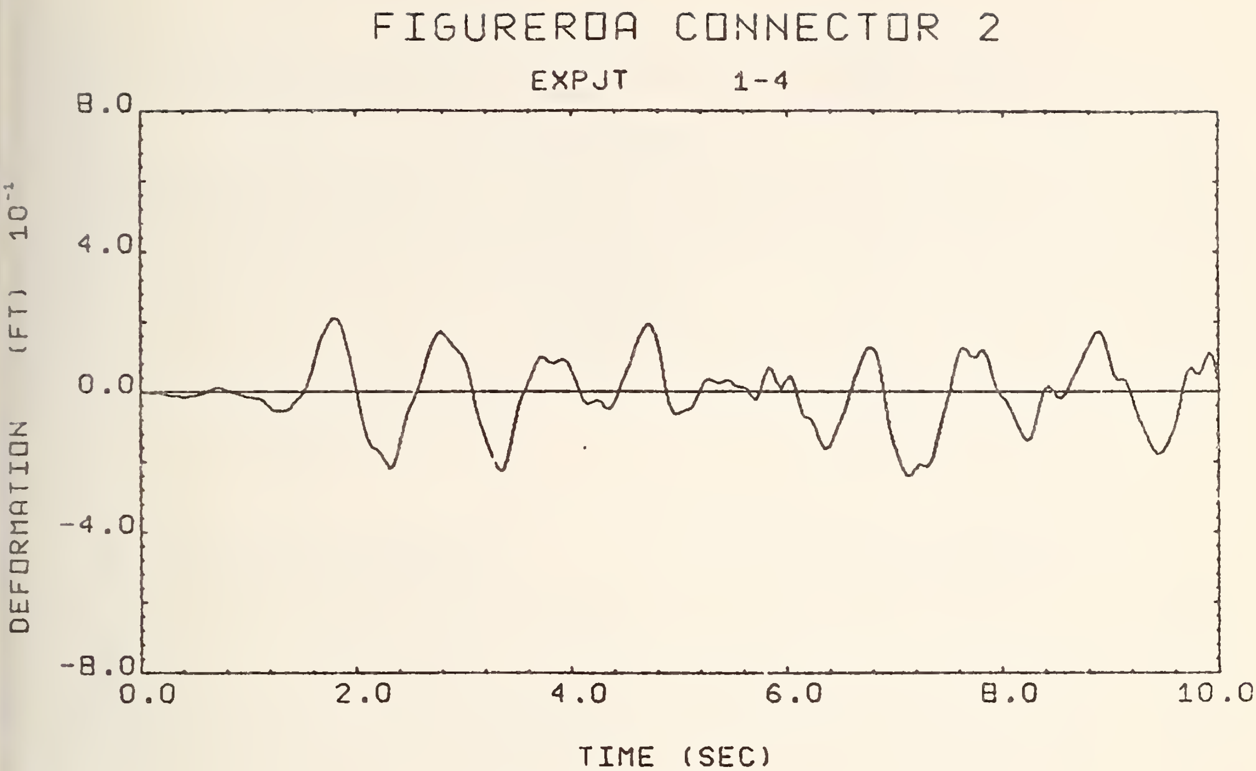
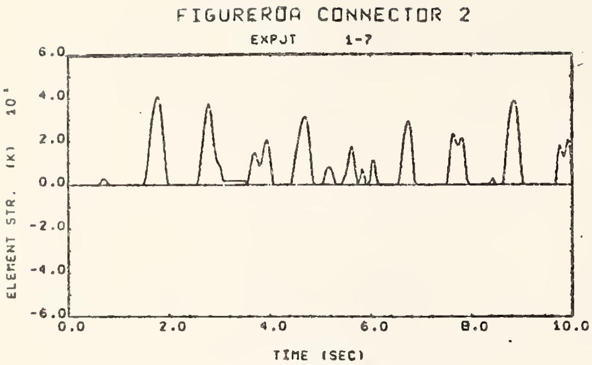


Fig. 81 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

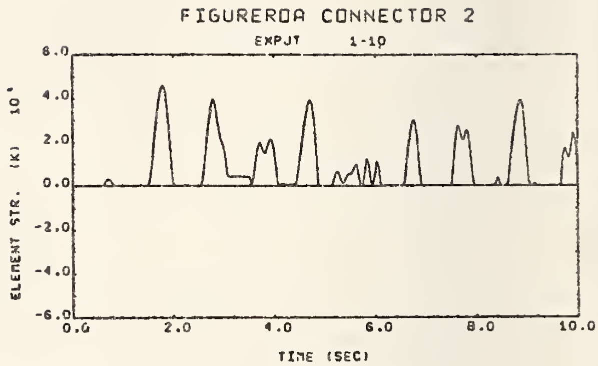


At the Outer Edge of the Deck

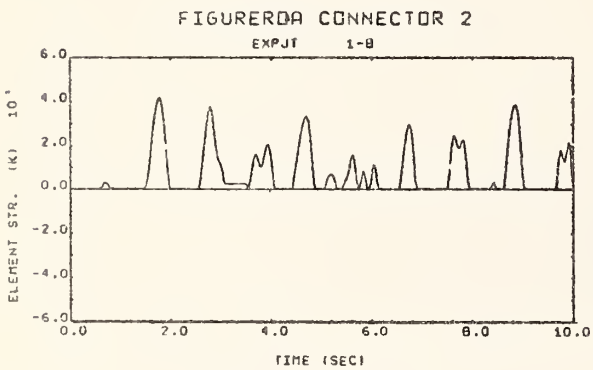
Fig. 82 Longitudinal Joint Separation at Expansion Joint # 1



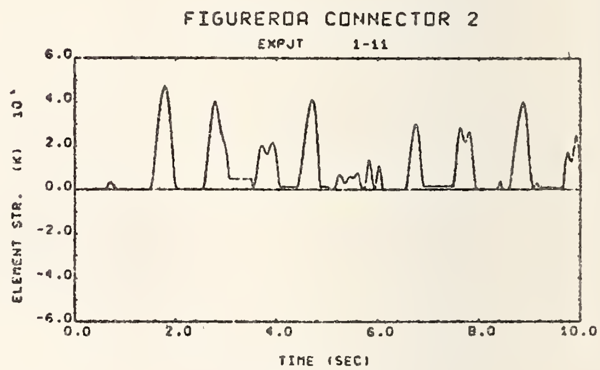
Tie Cable # 1



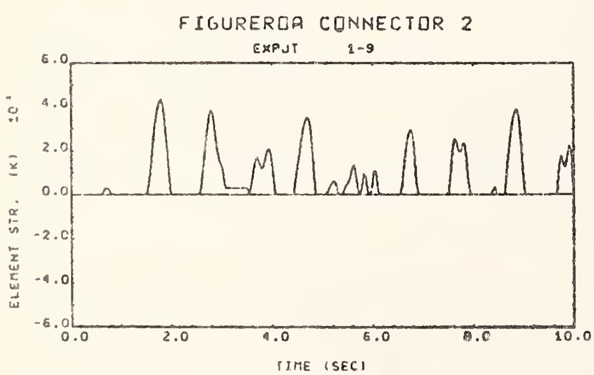
Tie Cable # 4



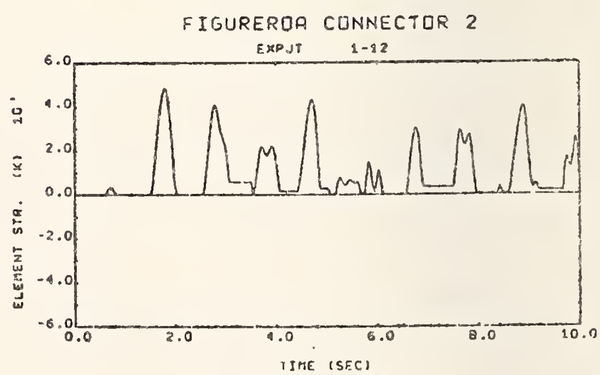
Tie Cable # 2



Tie Cable # 5



Tie Cable # 3



Tie Cable # 6

Fig. 83 Longitudinal Tie Cable Forces at Expansion Joint # 1

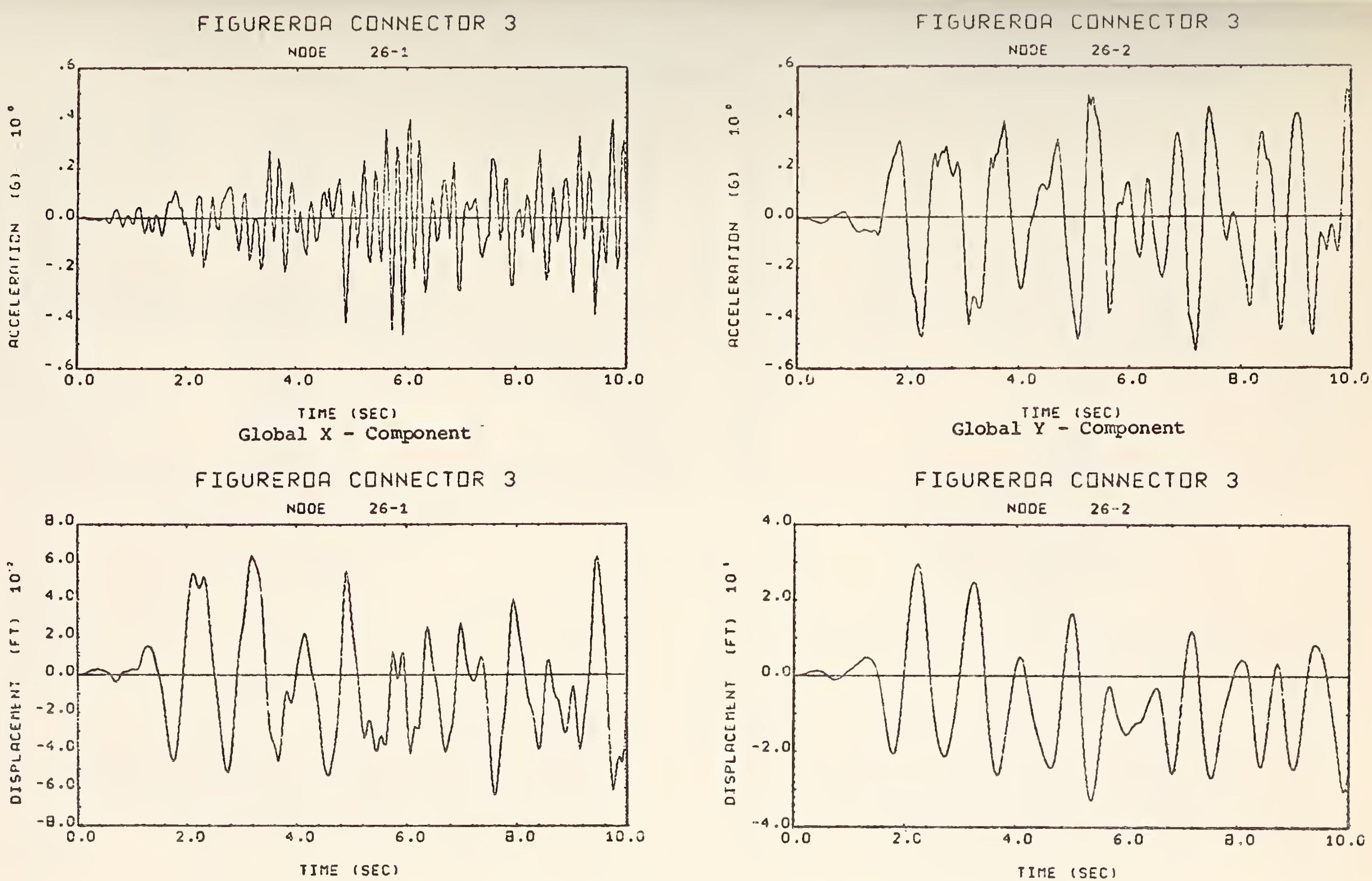
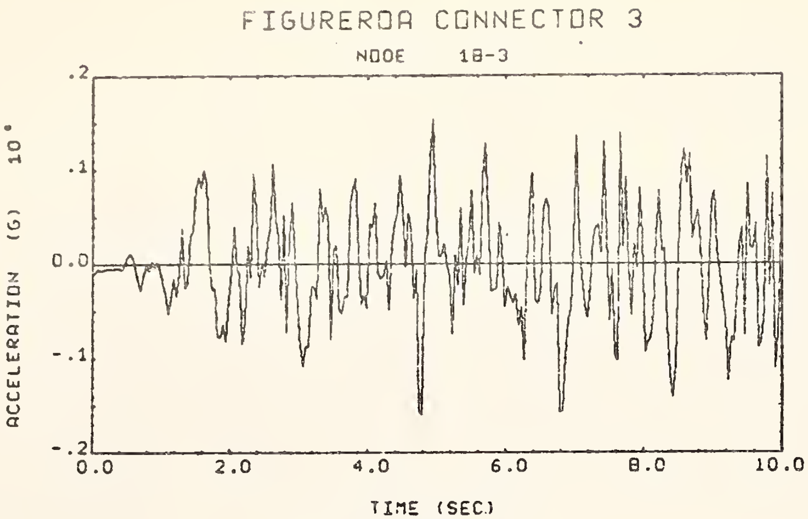
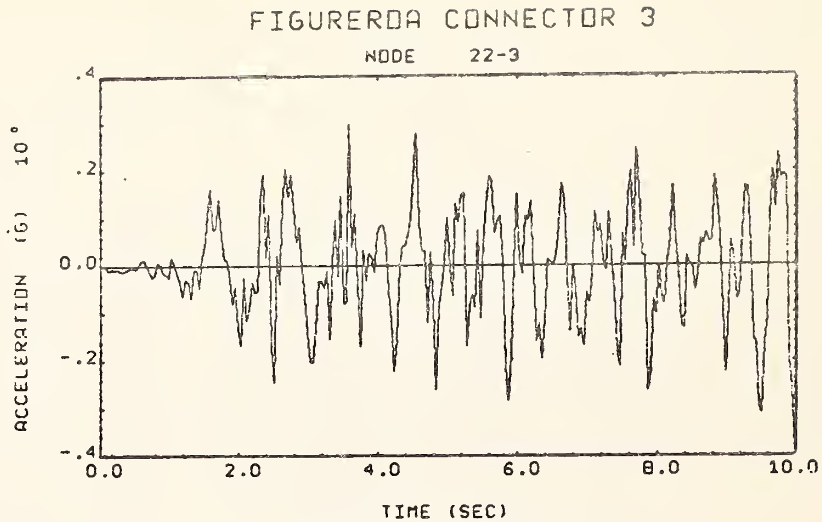


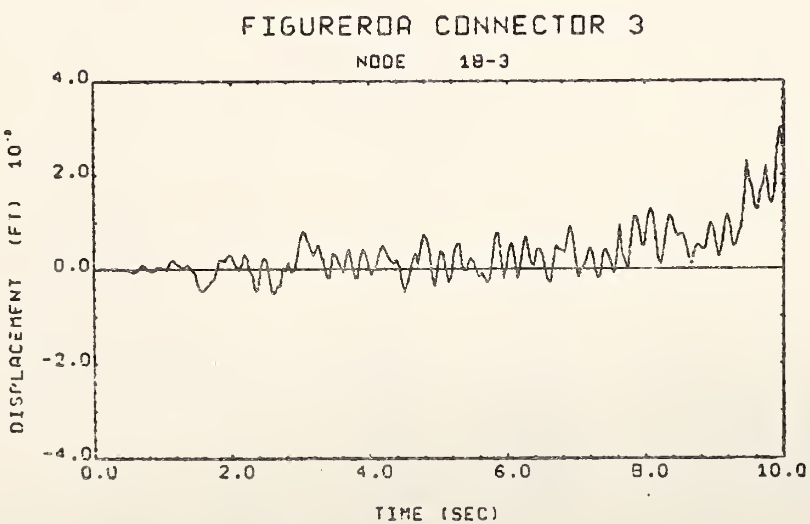
Fig. 84 Horizontal Accelerations and Displacements at the Top of Column # 3



Top of Column # 3



Center of Span # 3



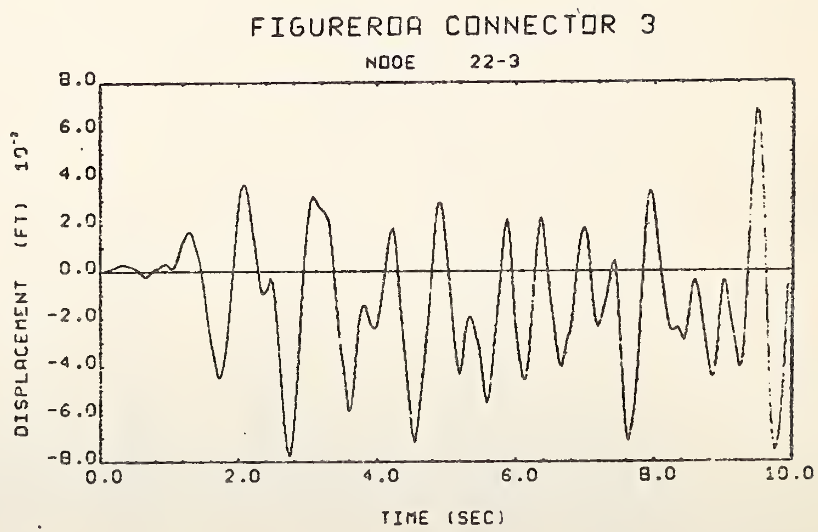
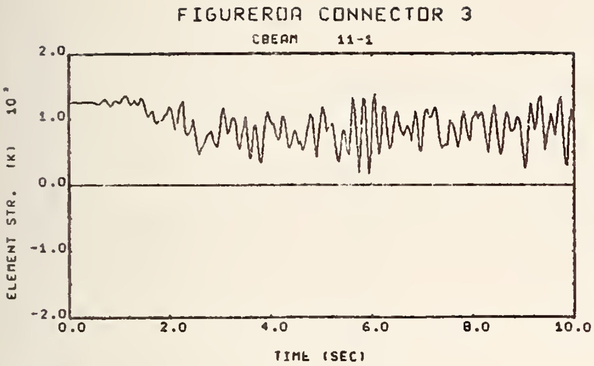
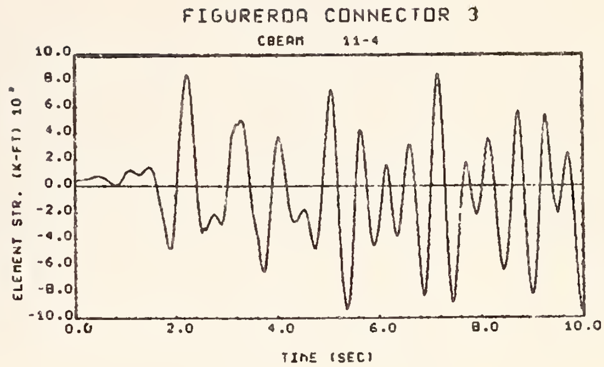


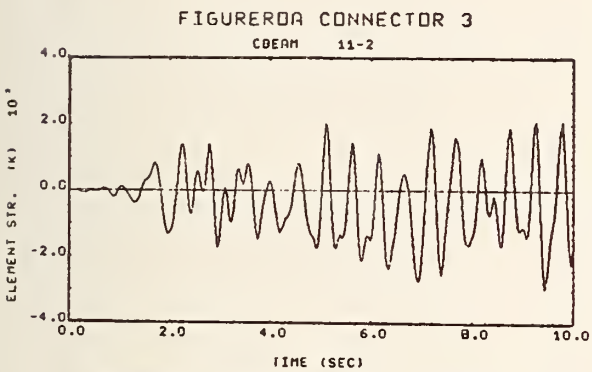
Fig. 85 Vertical Accelerations and Displacements at the Top of Column # 3 and the Center of Span # 3



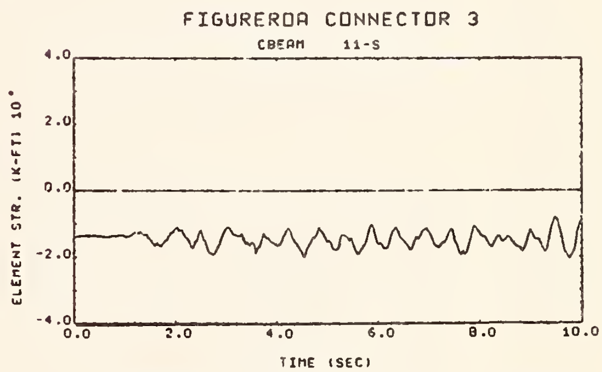
Axial Force



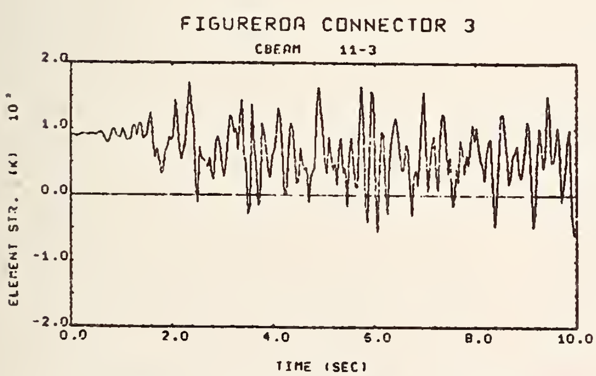
Torsional Moment



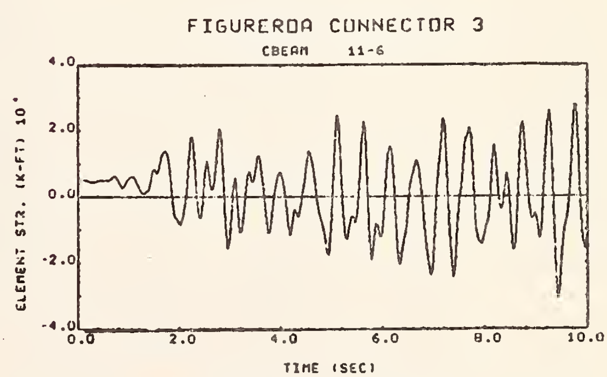
Shear in the Local y - Direction



Moment about the Local y - Axis

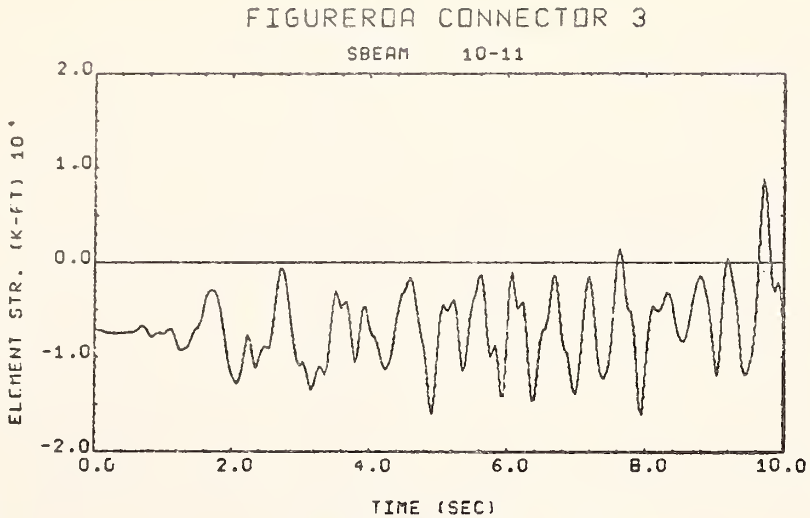


Shear in the Local z - Direction

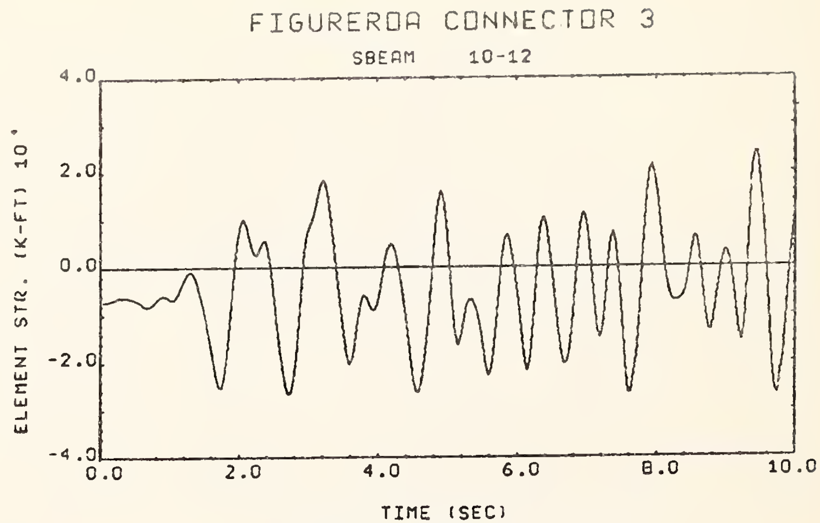


Moment about the Local z - Axis

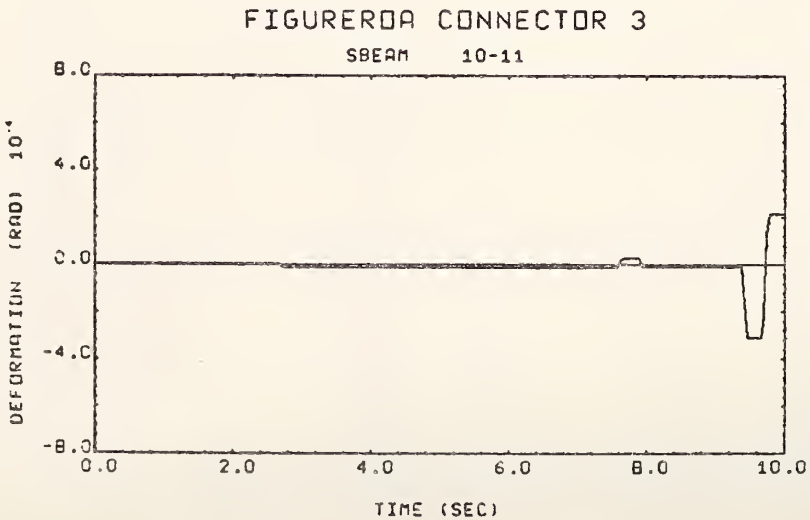
Fig. 86 Generalized Forces in the Girder at the Center of Span # 3



About the Local y - Axis



About the Local z - Axis



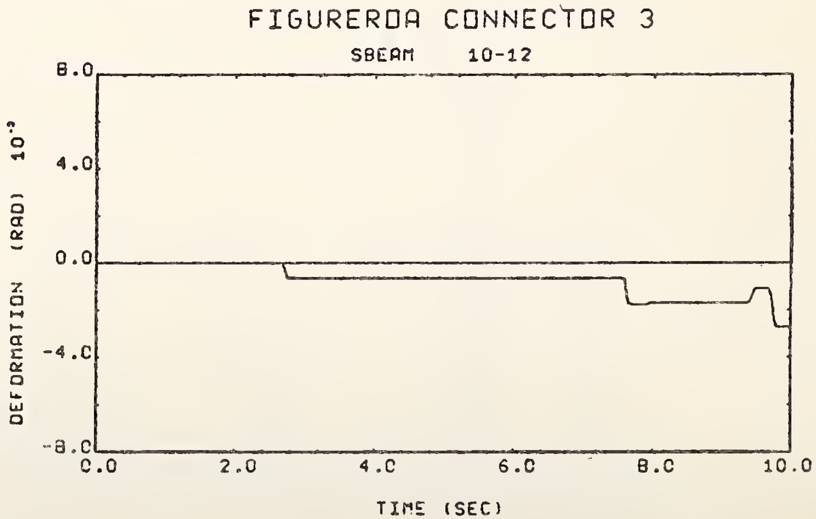
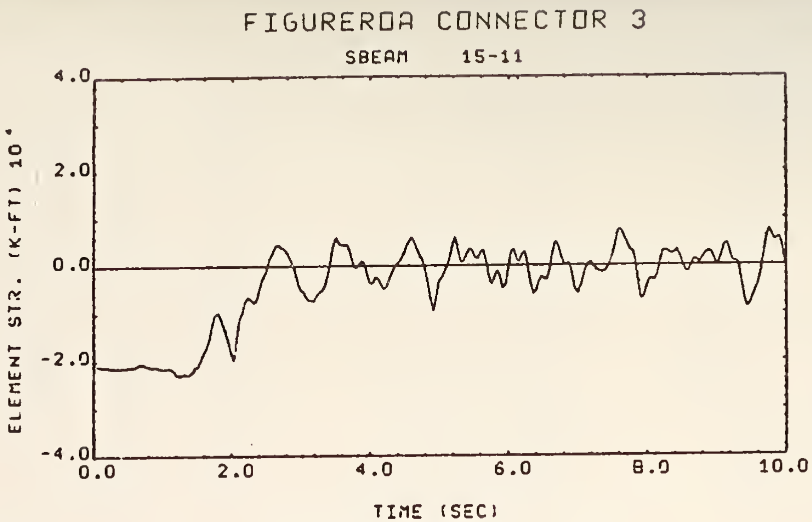
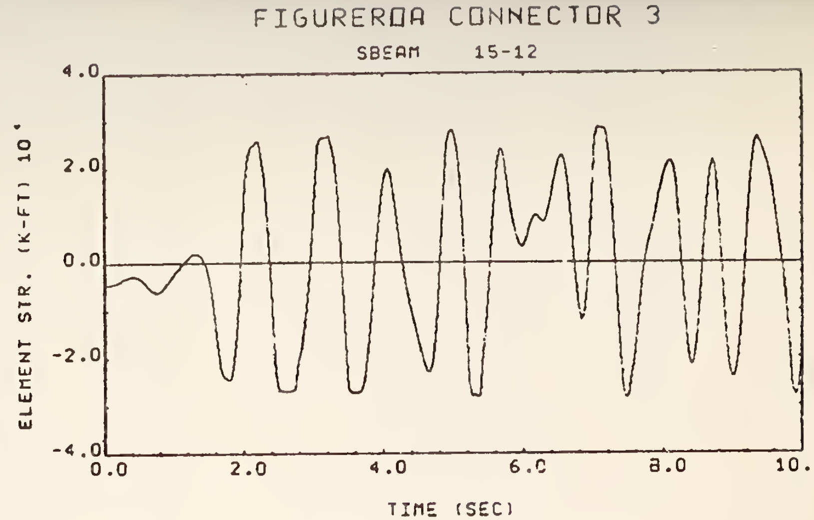


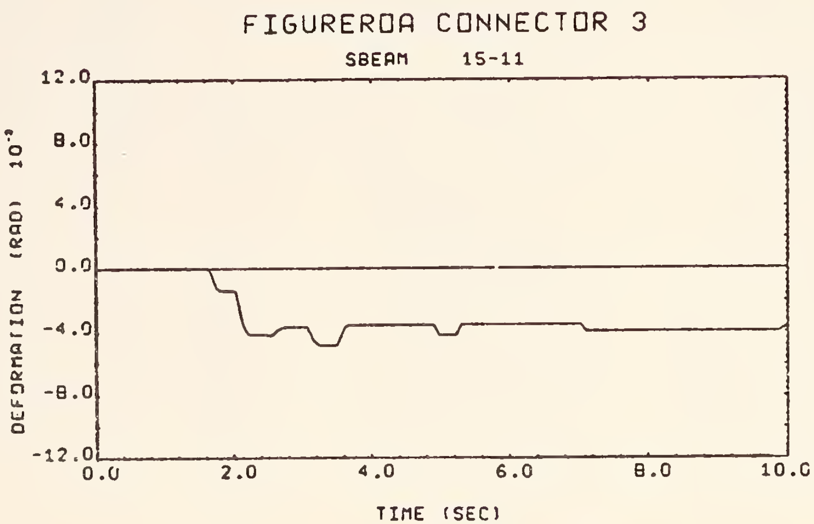
Fig. 87 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 3



About the Local y - Axis



About the Local z - Axis



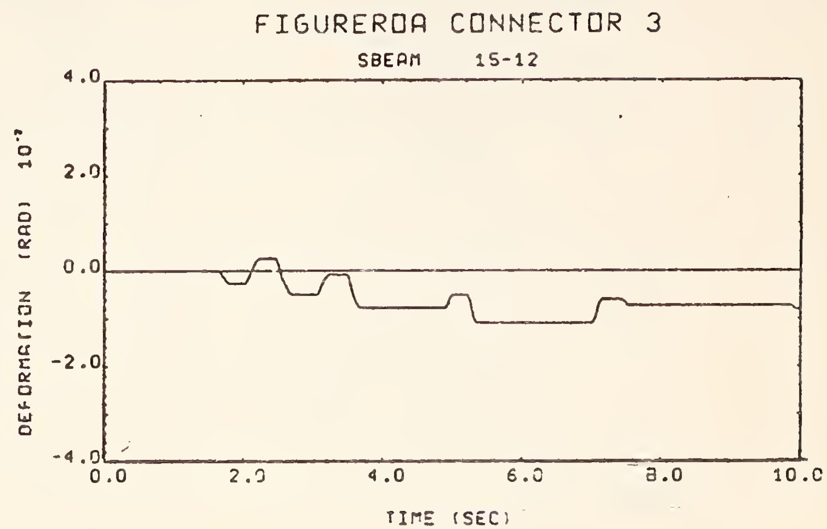
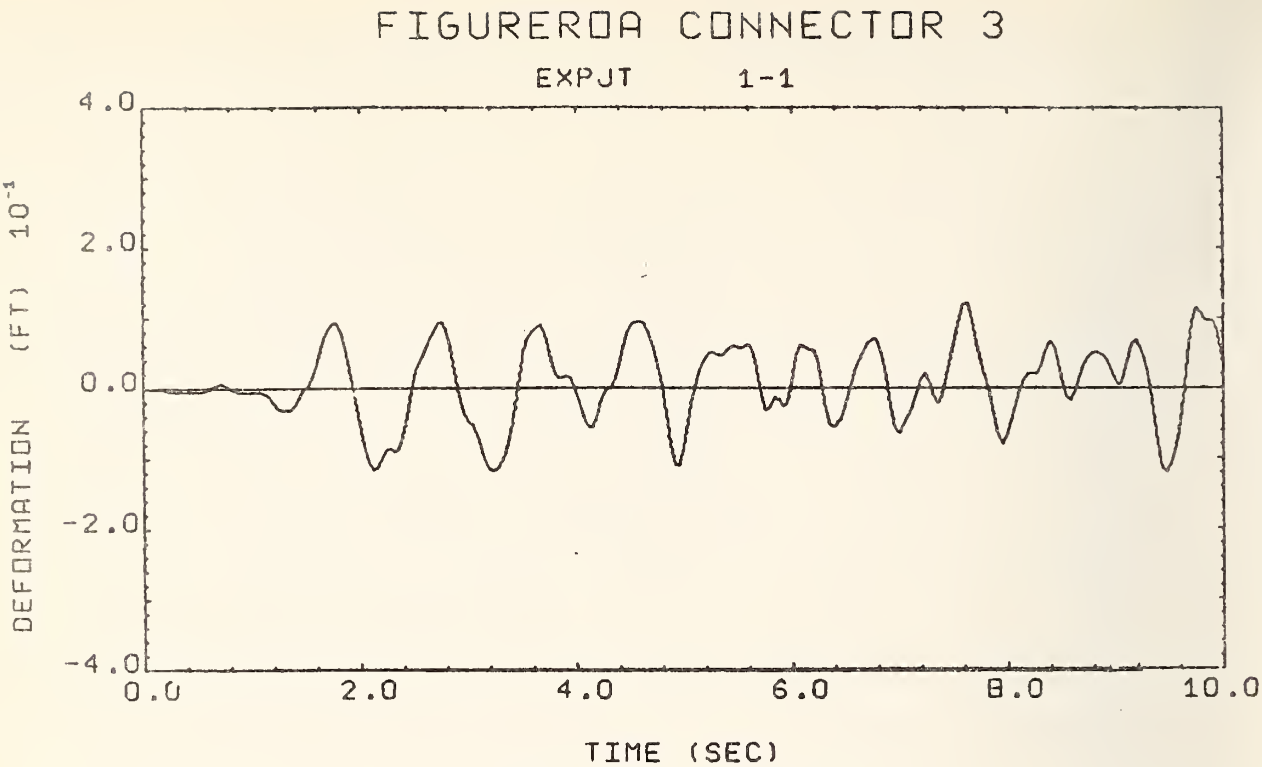
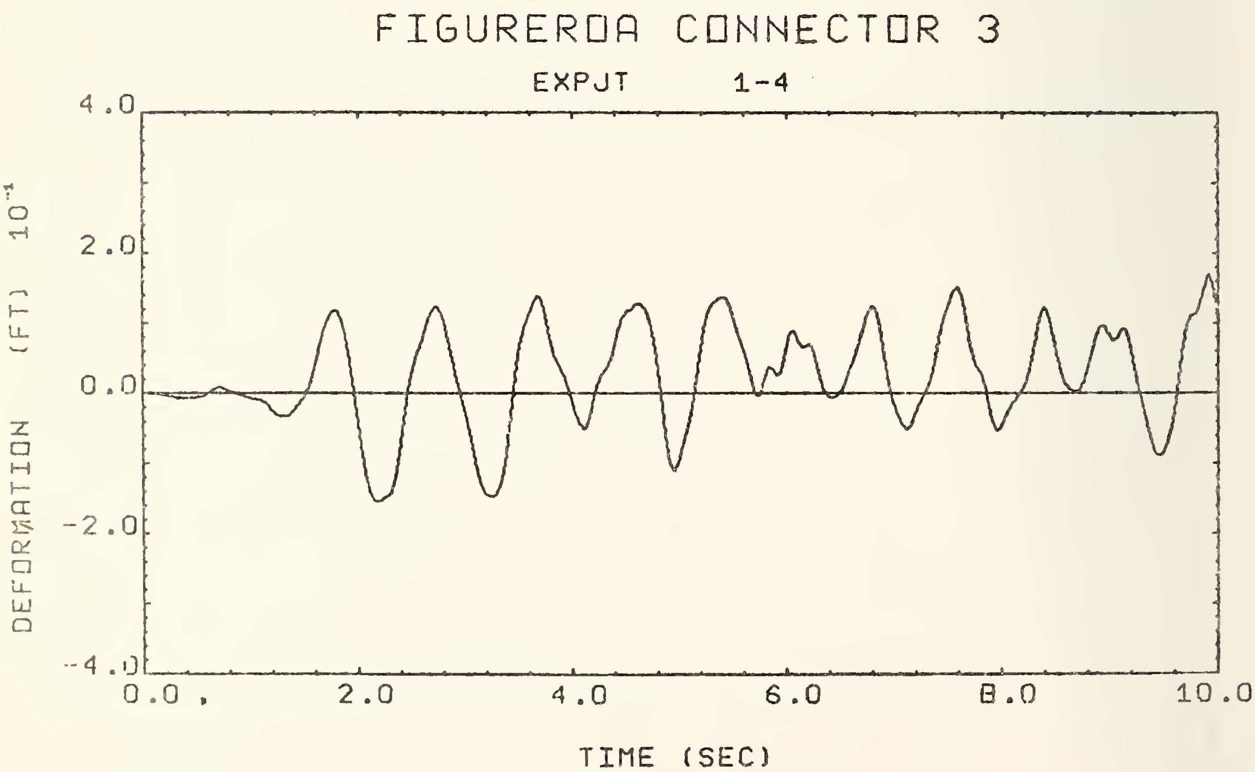


Fig. 88 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

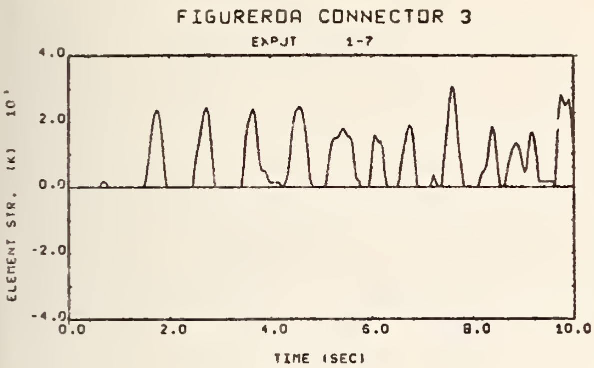


At Inner Edge of the Deck

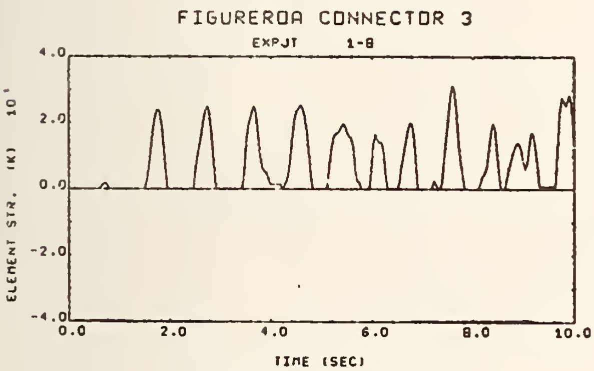


At Outer Edge of the Deck

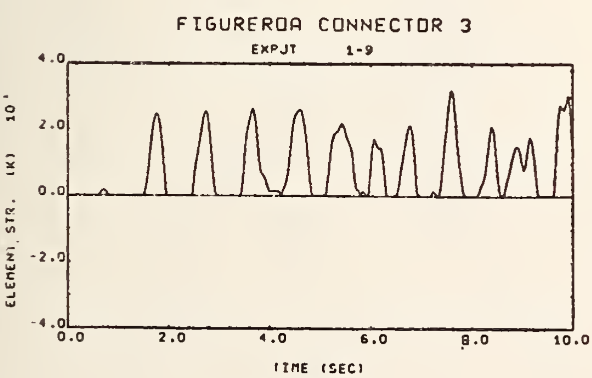
Fig. 89 Longitudinal Joint Separations at Expansion Joint # 1



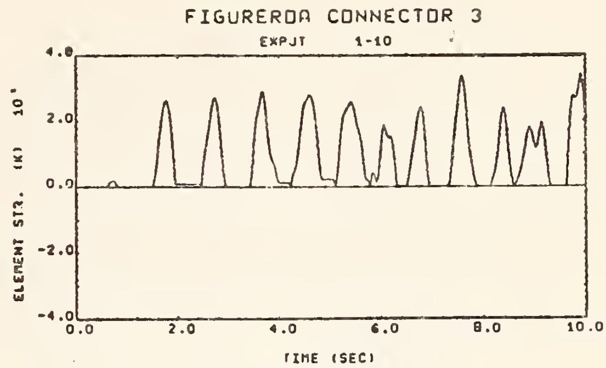
Tie Cable # 1



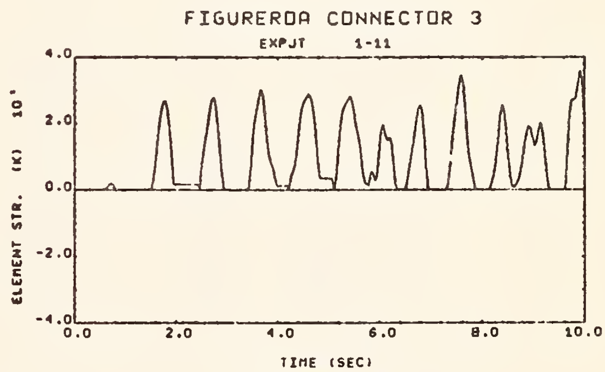
Tie Cable # 2



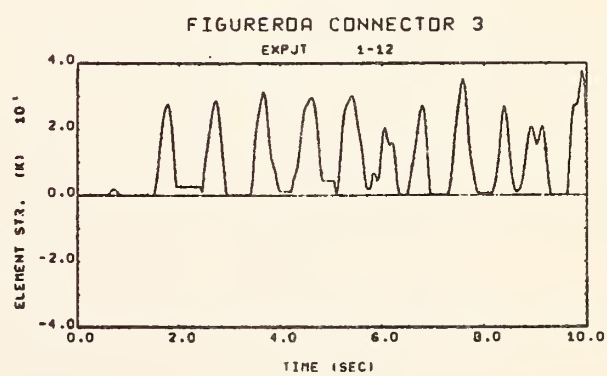
Tie Cable # 3



Tie Cable # 4



Tie Cable # 5



Tie Cable # 6

Fig. 90 Longitudinal Tie Cable Forces at Expansion Joint # 1

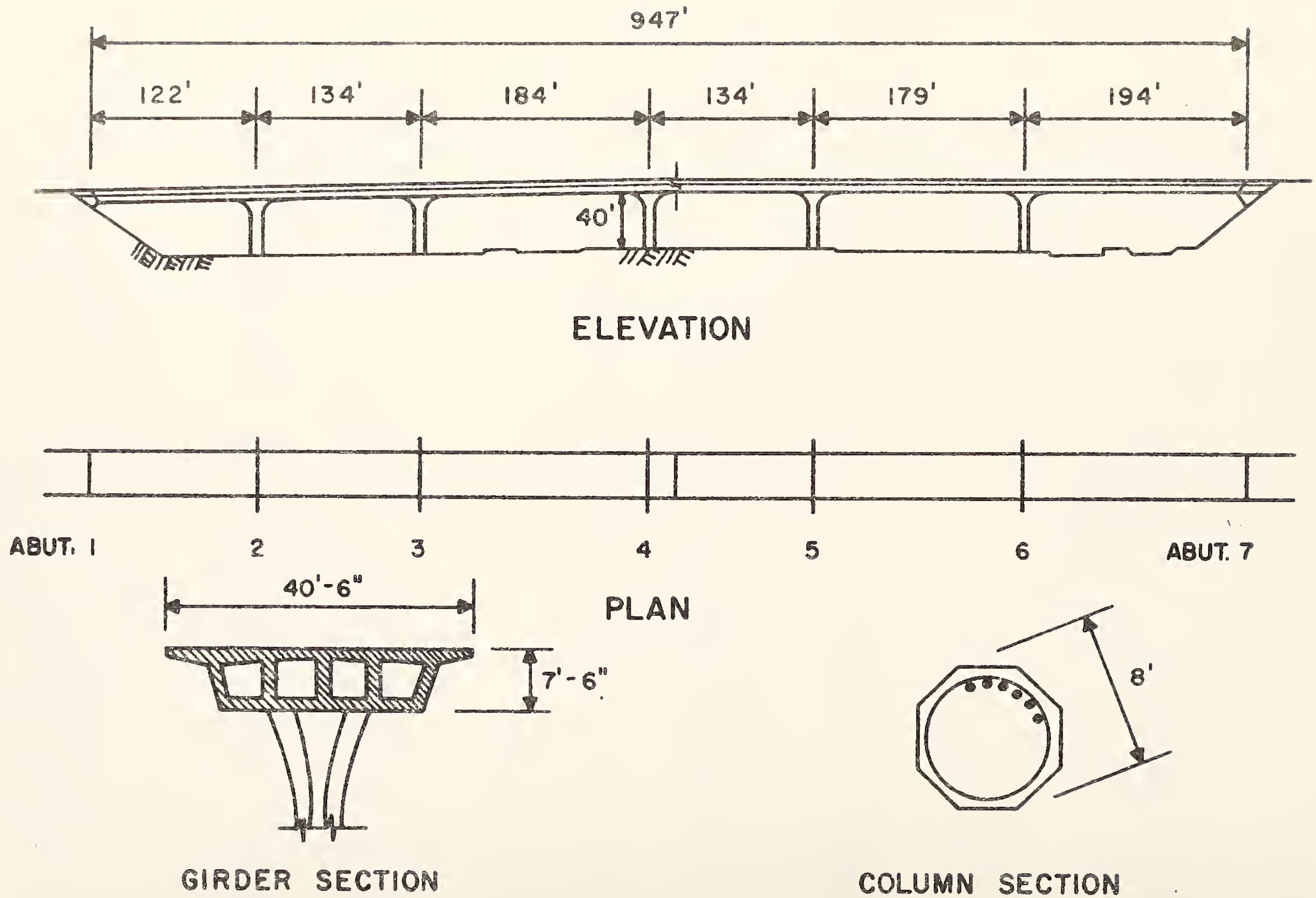


Fig. 91 The Structural System of the Straight Figueroa Street U. C. Connector

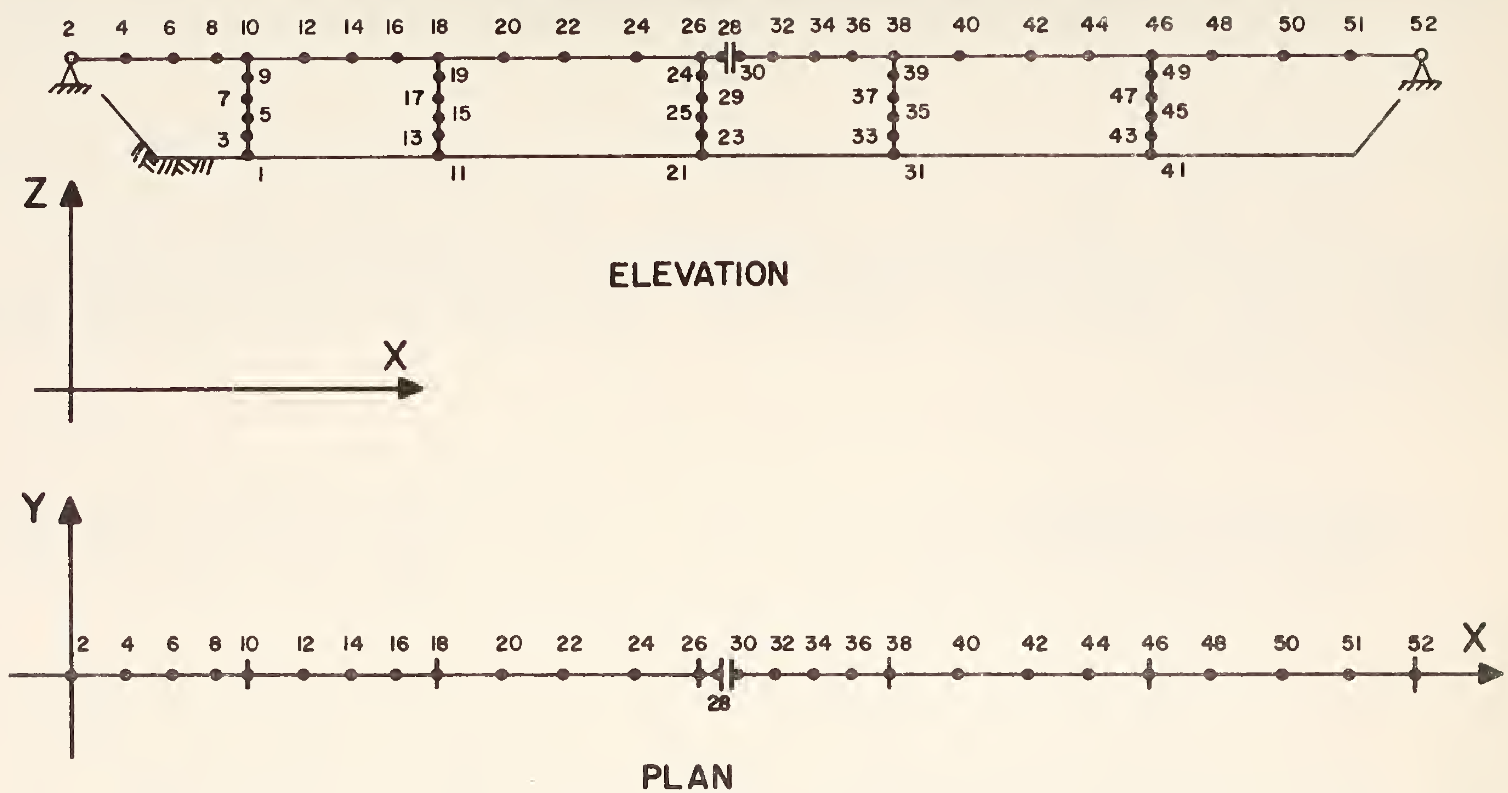


Fig. 92 Lumped Parameter System of the Straight Figueroa Street U. C. Connector

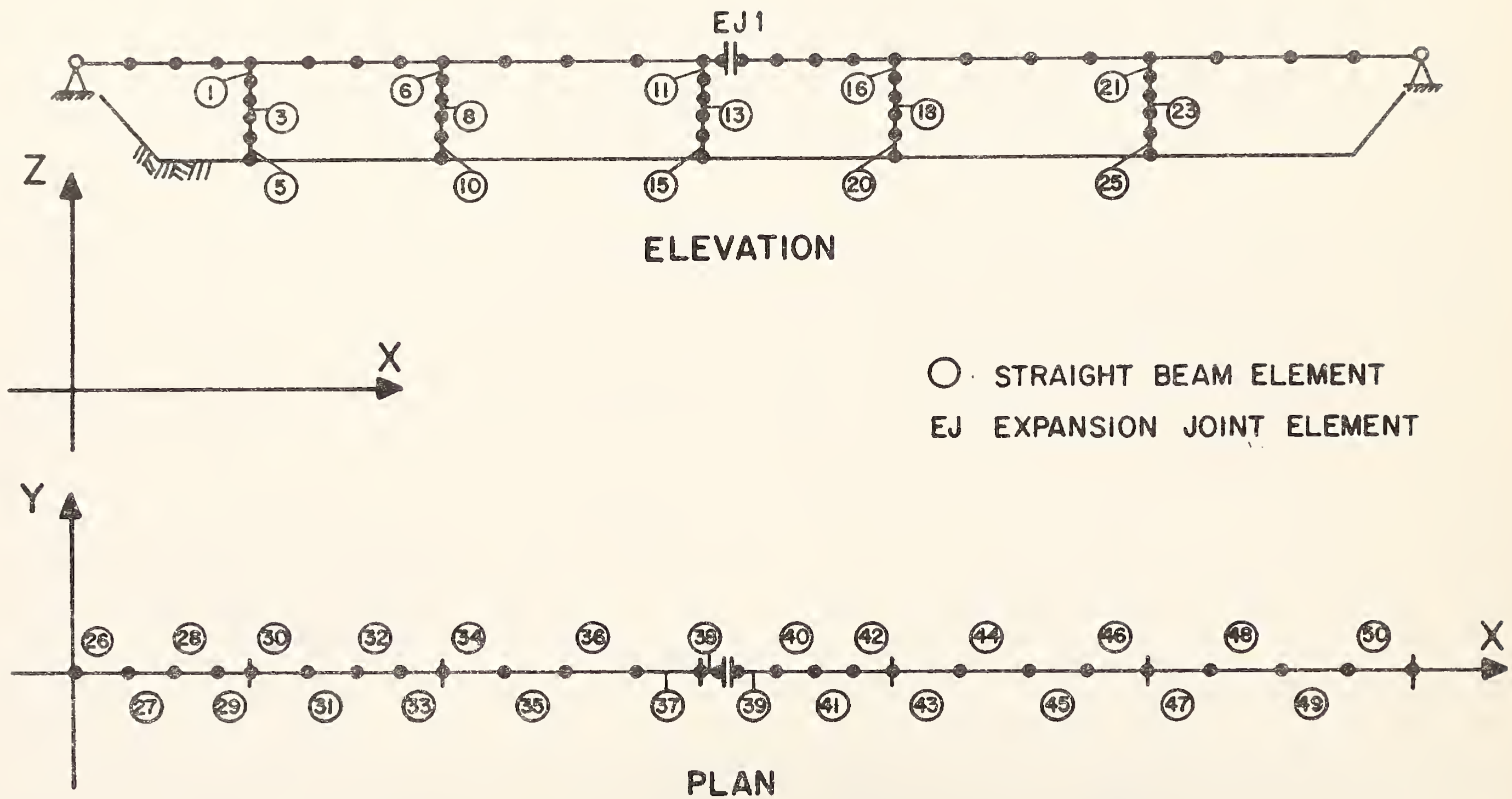


Fig. 93 Finite Element Model of Straight Figueroa Street U. C. Connector

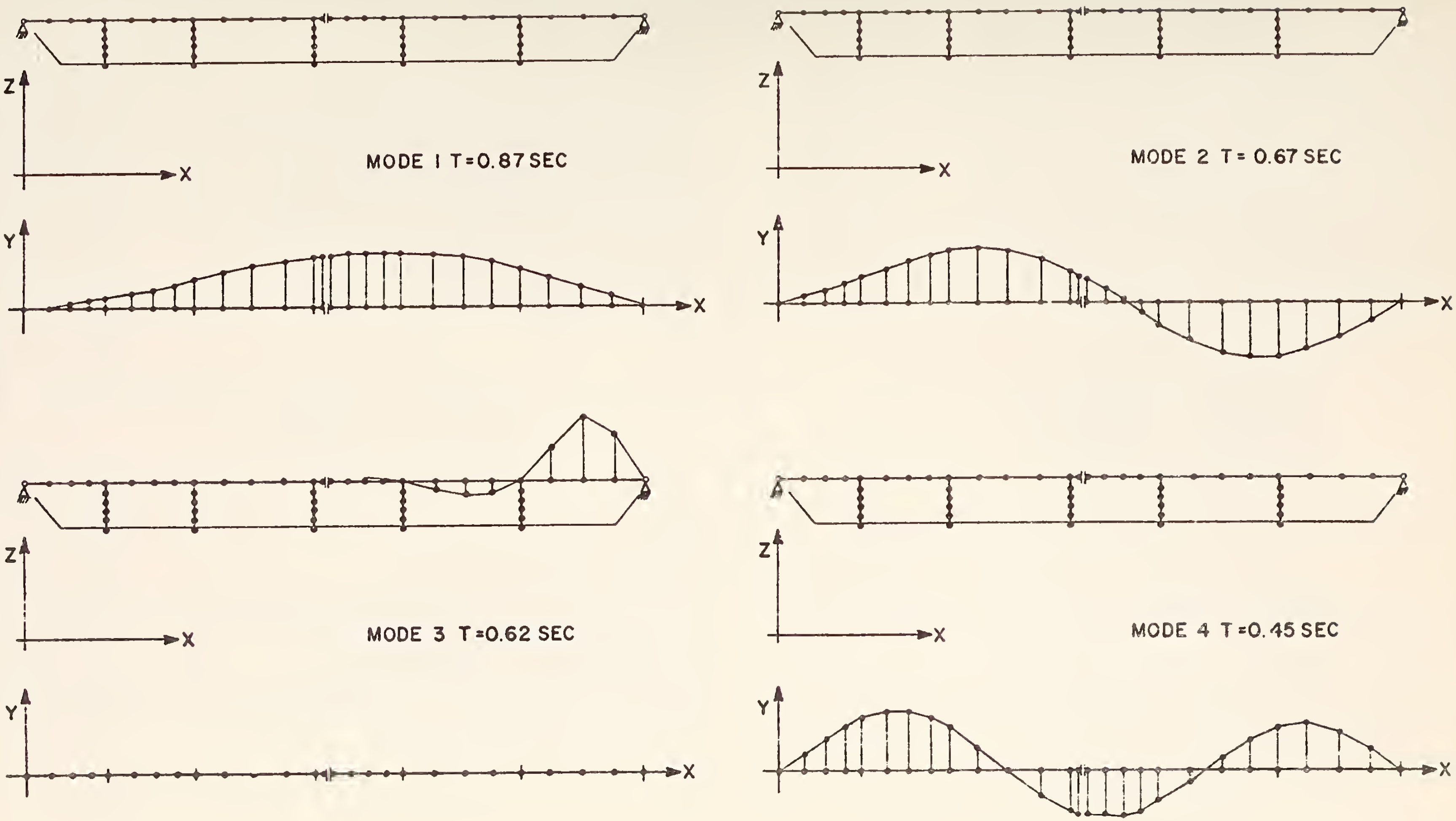


Fig. 94 Mode Shapes of the Straight Figueroa Street Undercrossing Connector: Zero Friction at the Expansion Joint

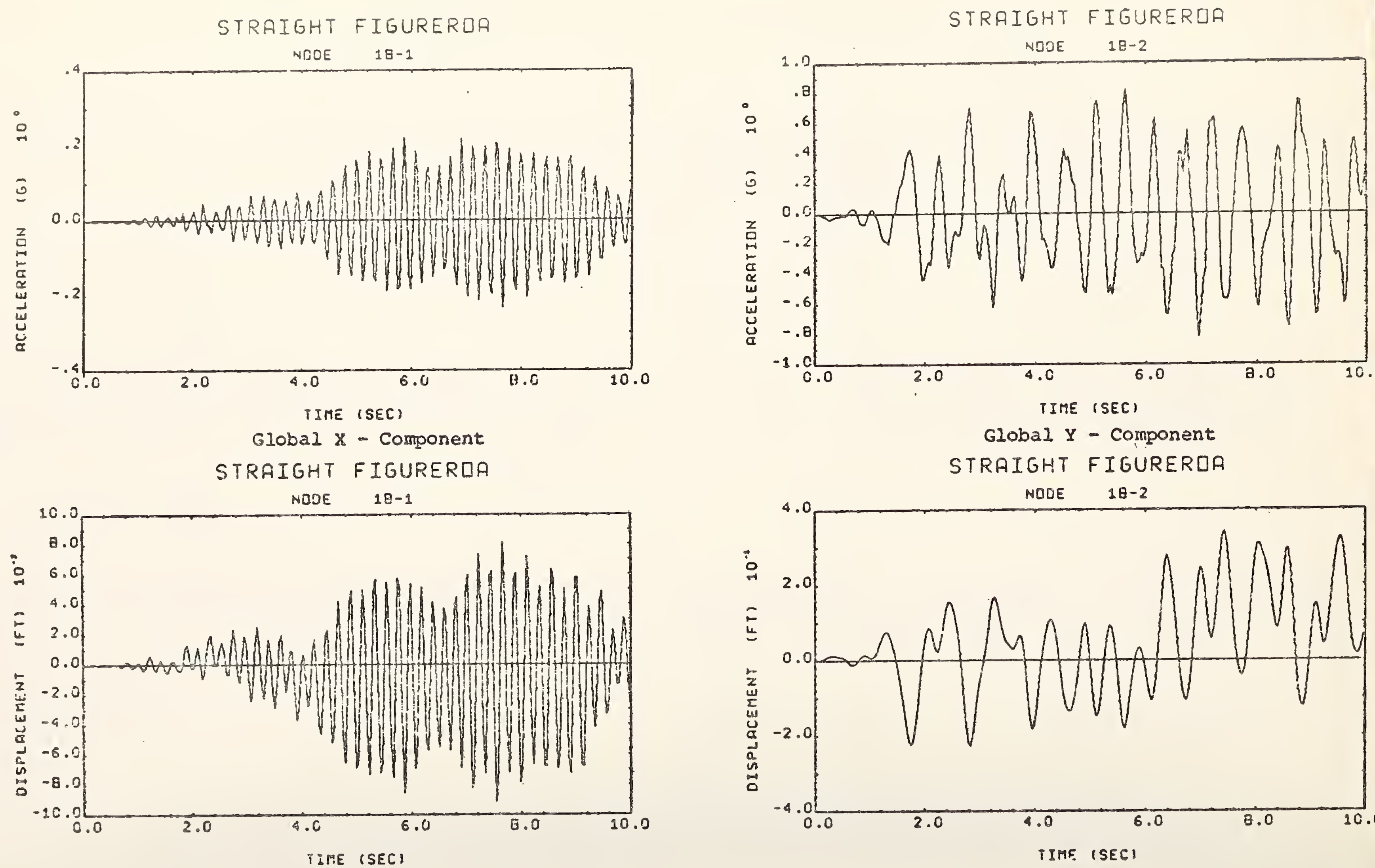


Fig. 95 Horizontal Accelerations and Displacements at the Top of Column # 3

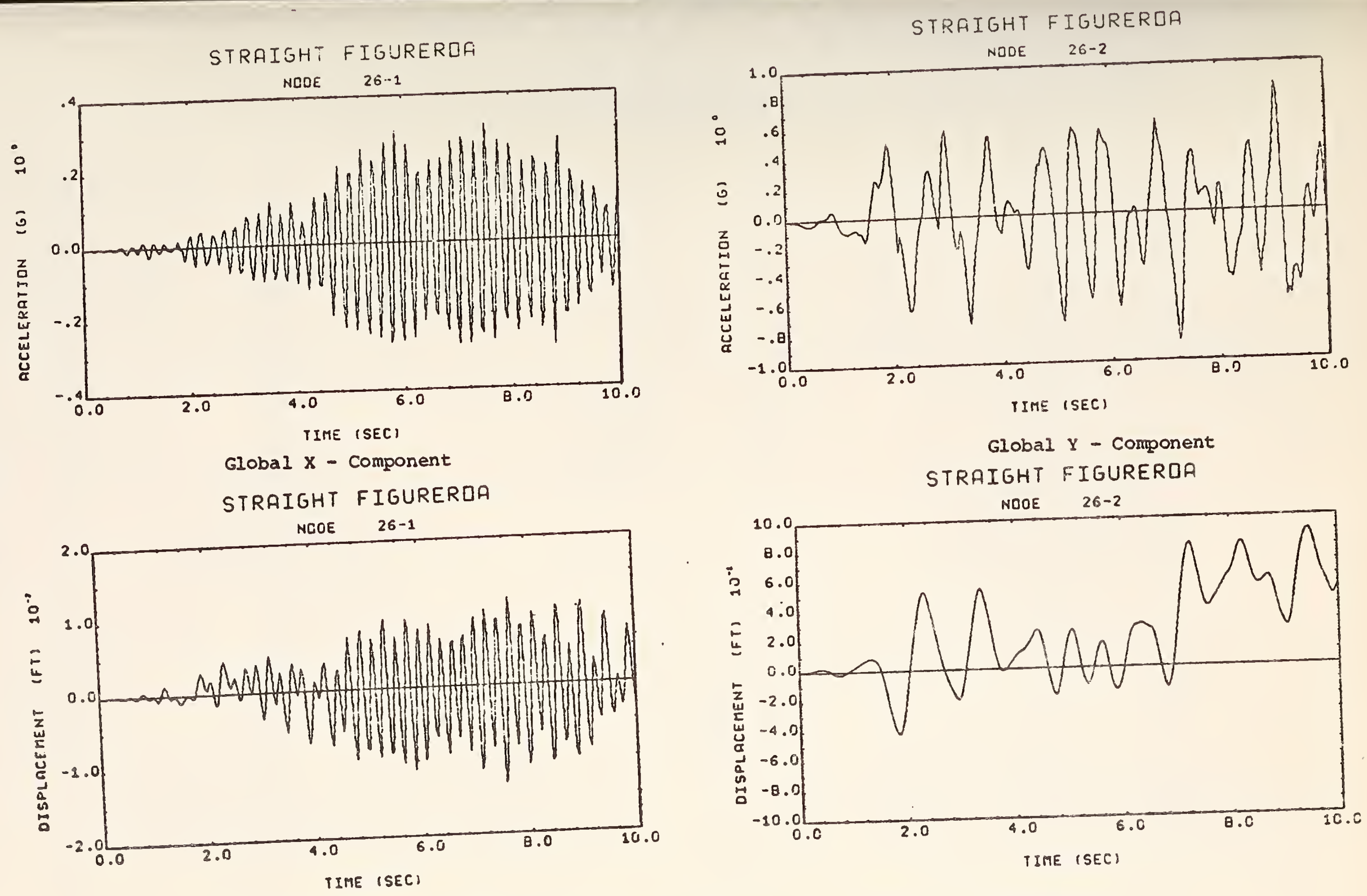
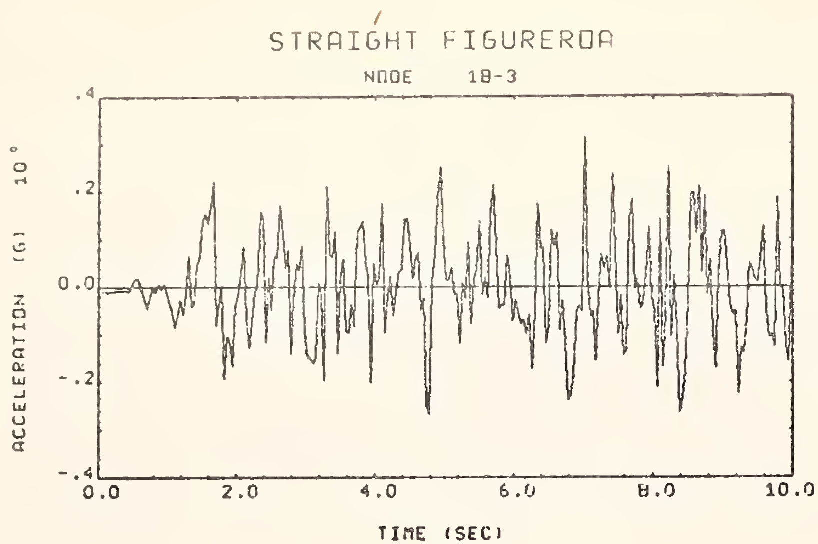
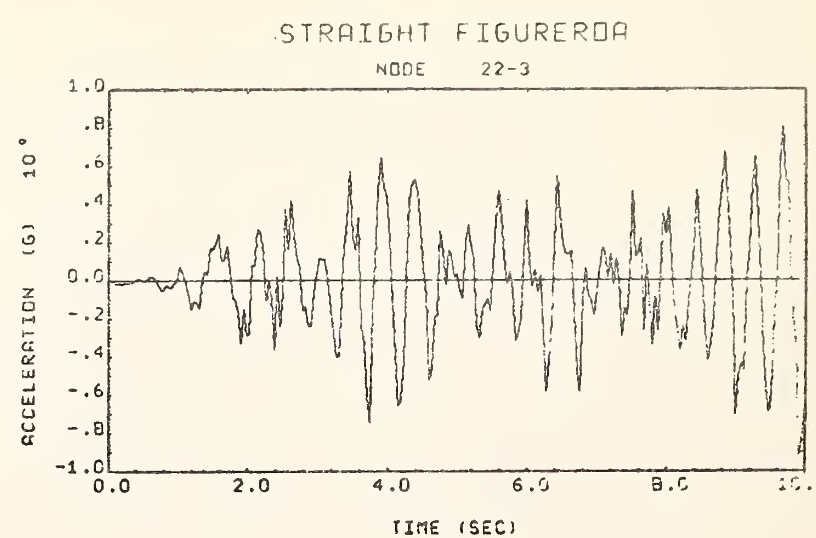


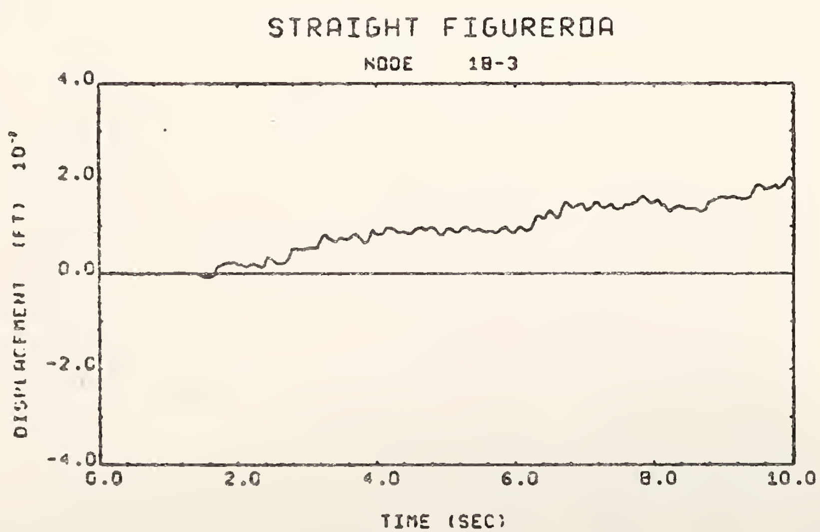
Fig. 96 Horizontal Accelerations and Displacements at the Top of Column # 4





Tie Column # 3

Center of Span # 3



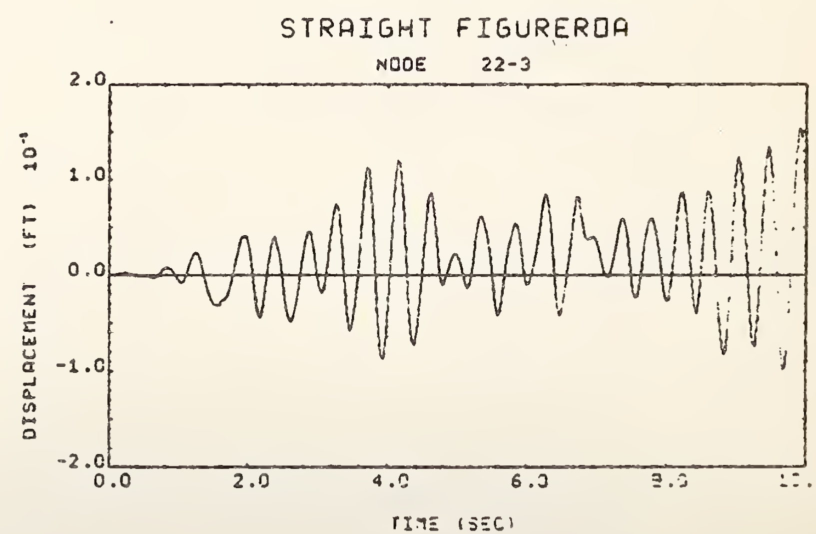
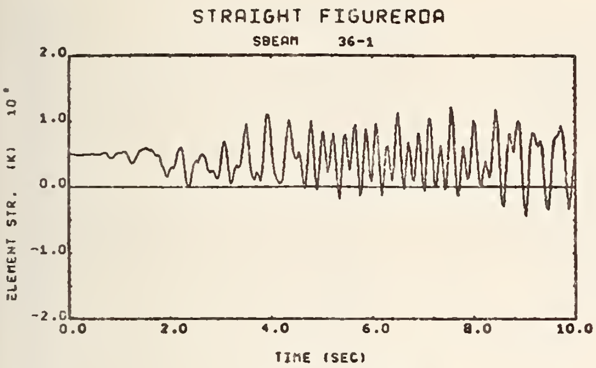
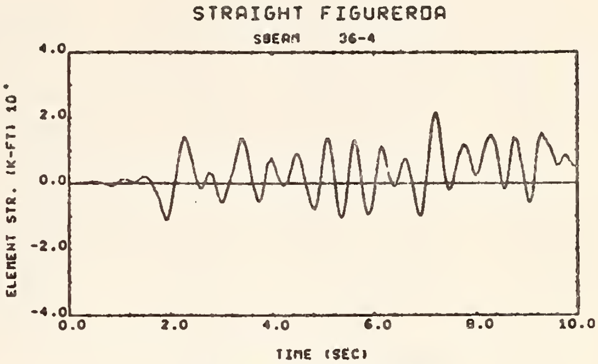


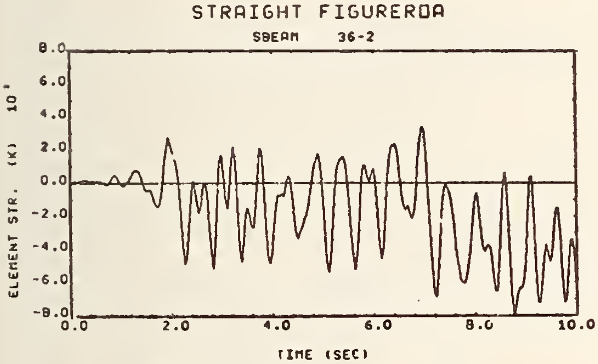
Fig. 97 Vertical Accelerations and Displacements at the Top of Column # 3 and the Center of Span # 3



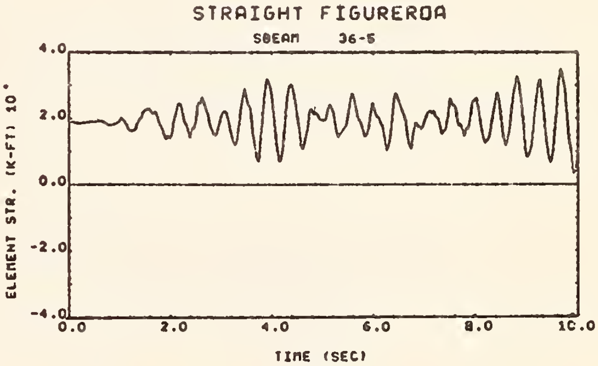
Axial Force



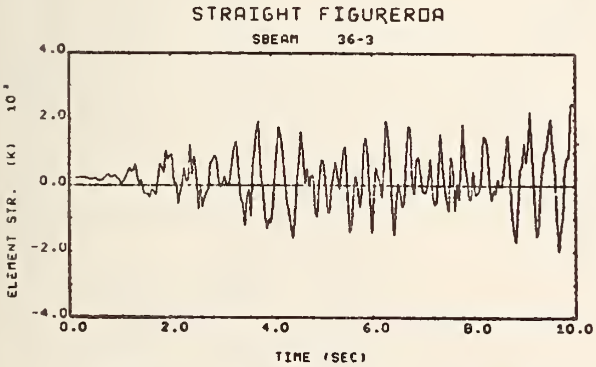
Torsional Moment



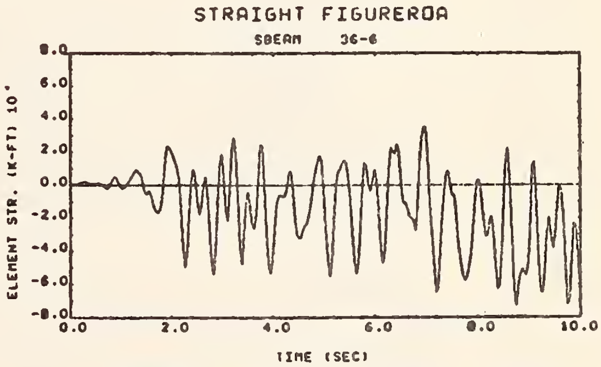
Shear in the Local y - Direction



Moment about the Local y - Axis



Shear in the Local z - Direction



Moment about the Local z - Axis

Fig. 98 Generalized Forces in the Girder at the Center of Span # 3

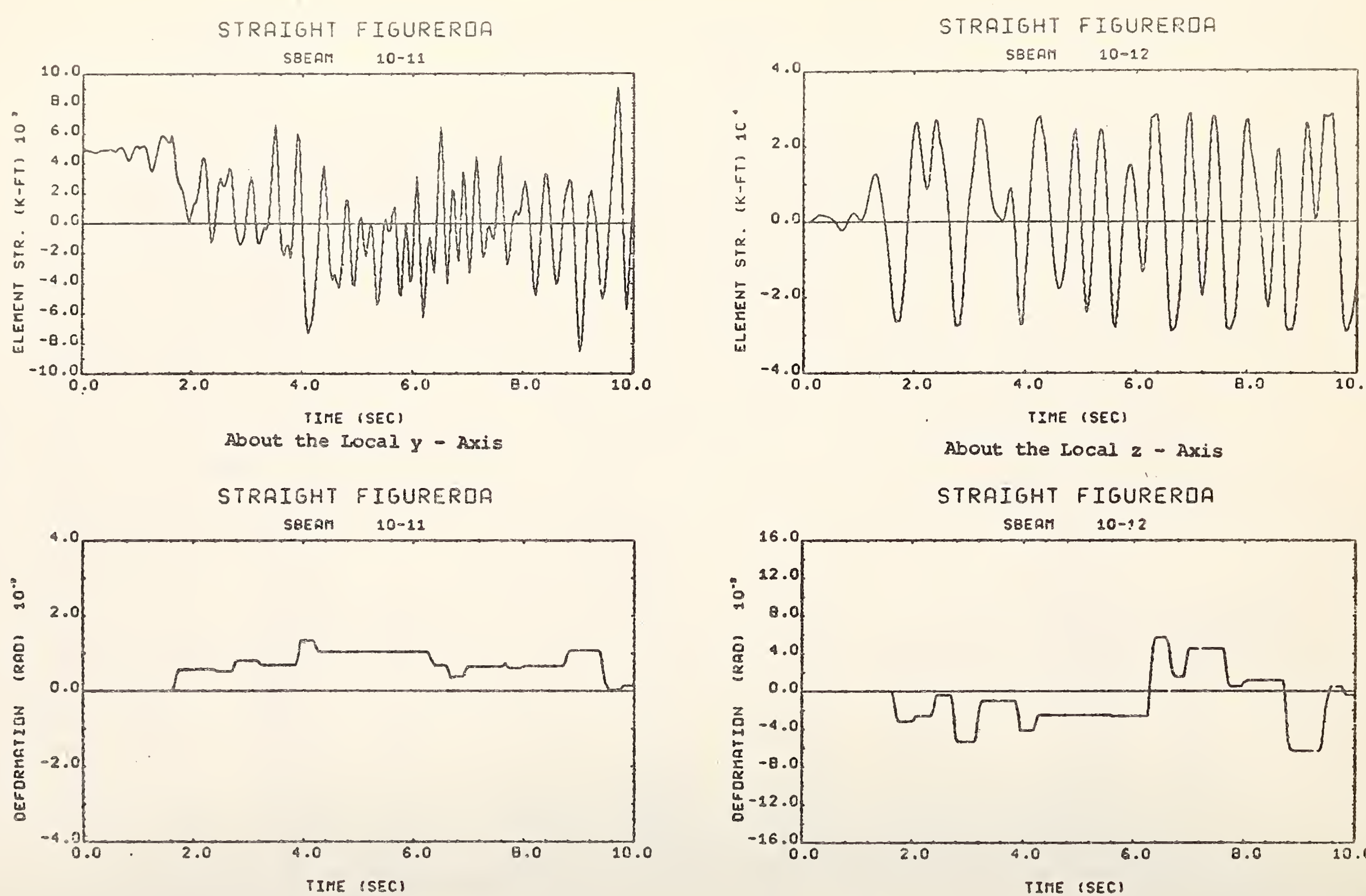


Fig. 99 Bending Moments and the Corresponding Plastic Rotations at the Base of Colum # 3

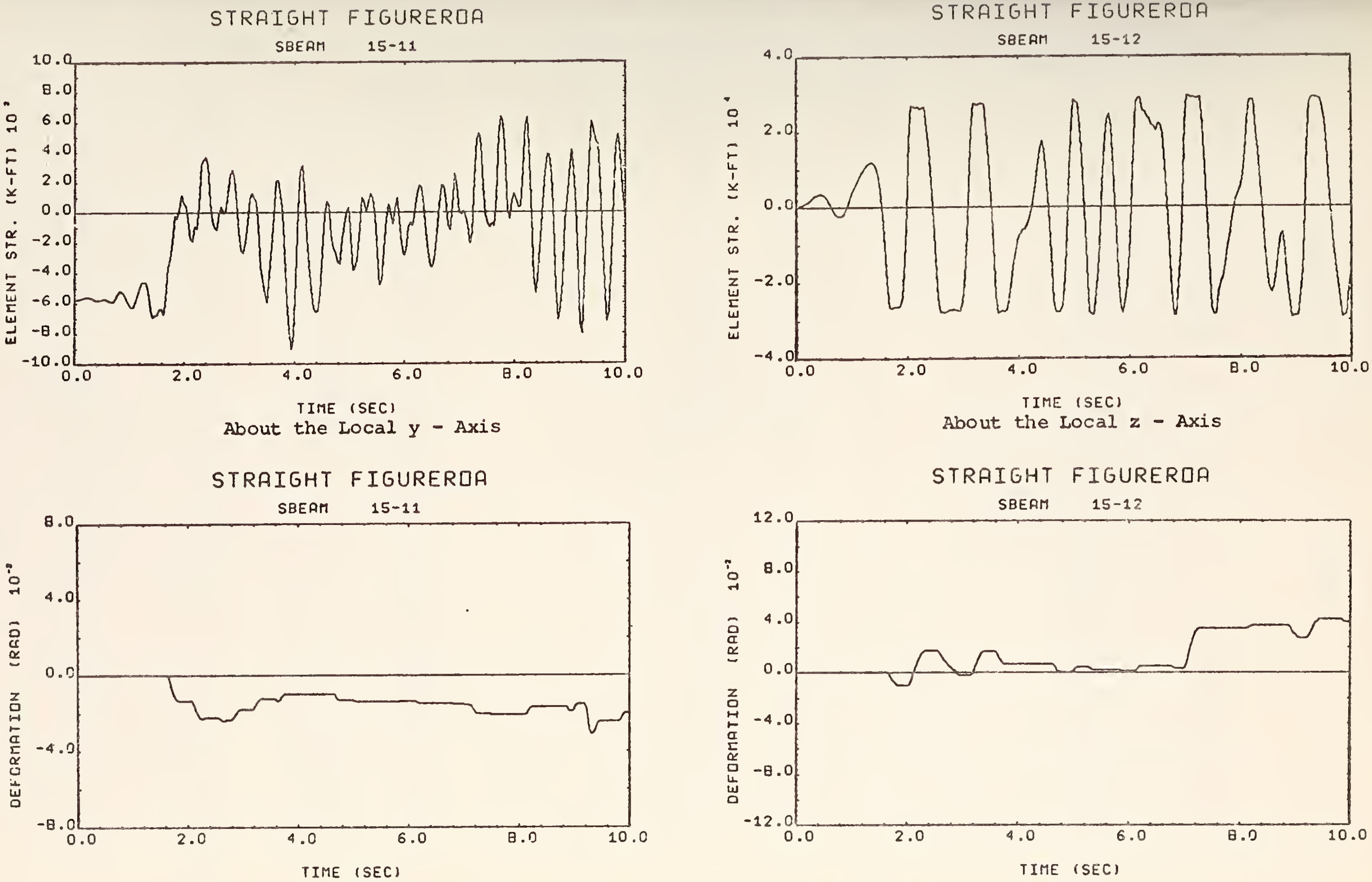
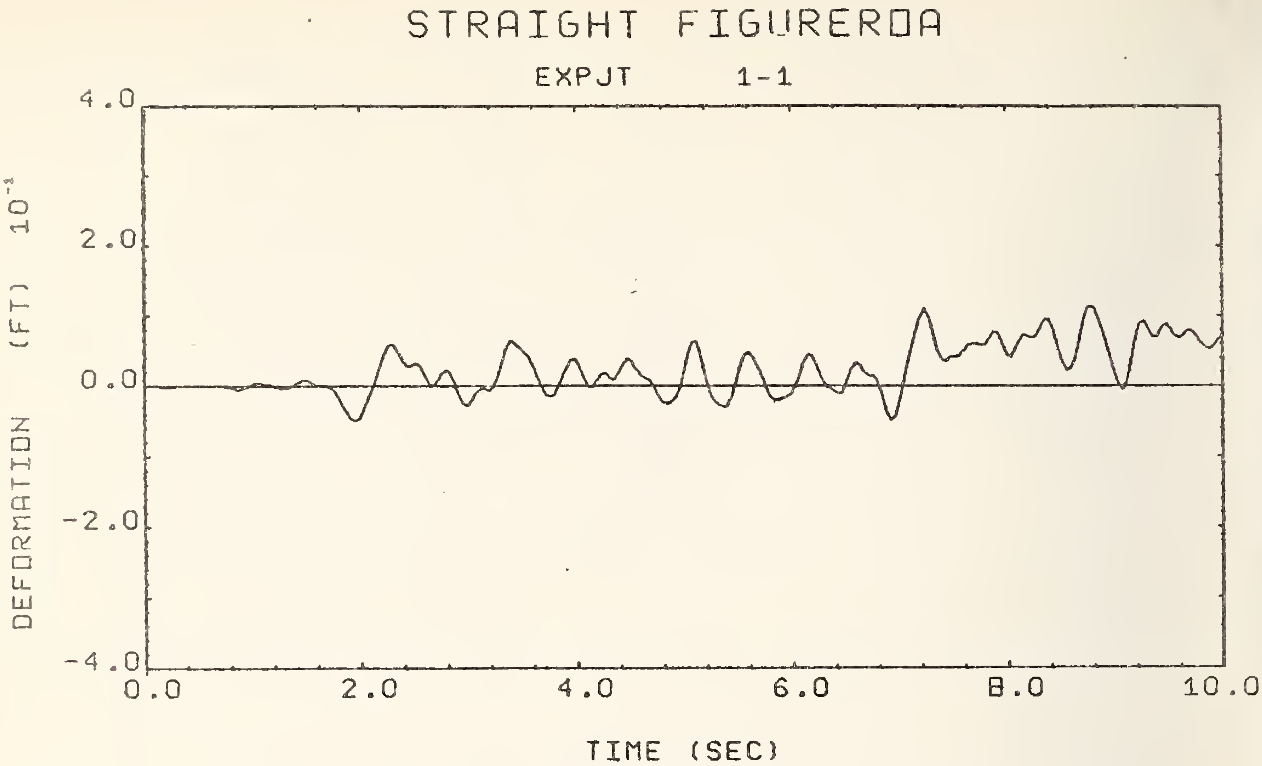
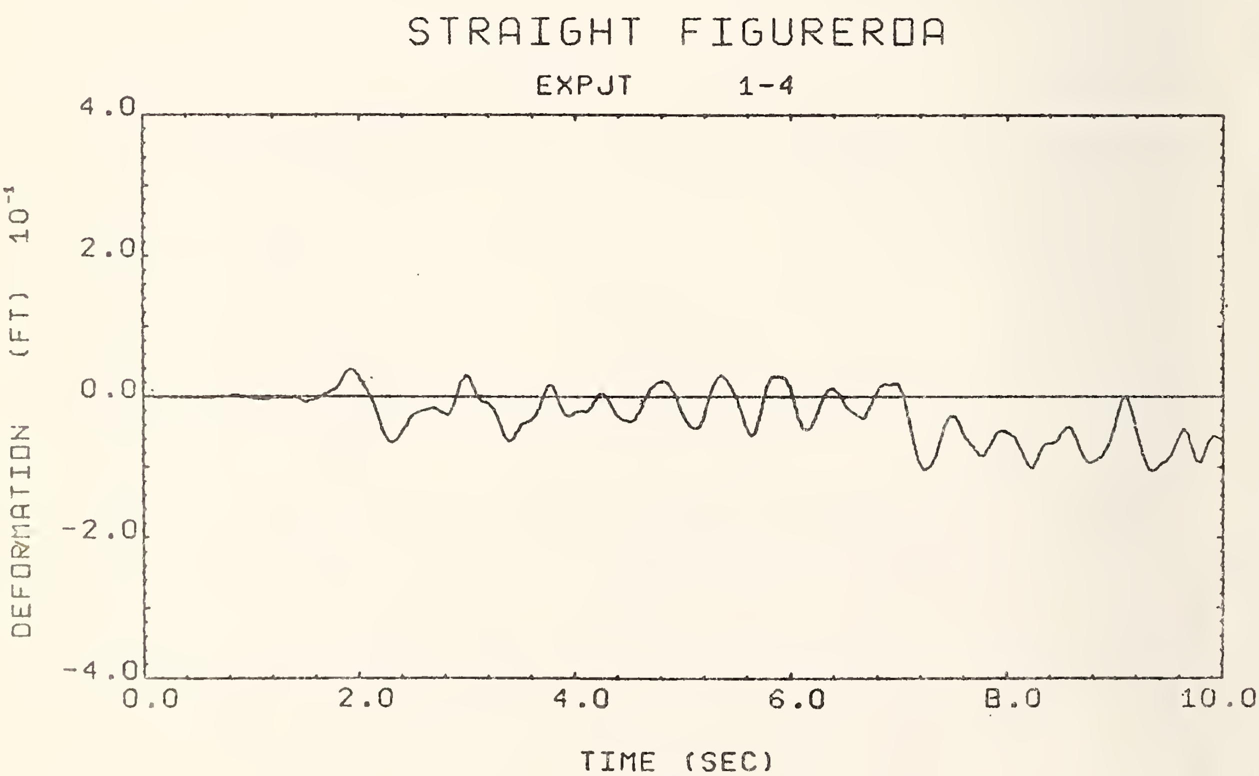


Fig. 100 Bending Moments and the Corresponding Plastic Rotations at the Base of Column # 4

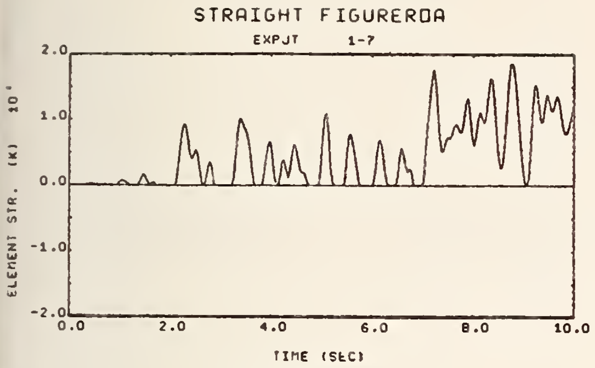


At Contact Point A

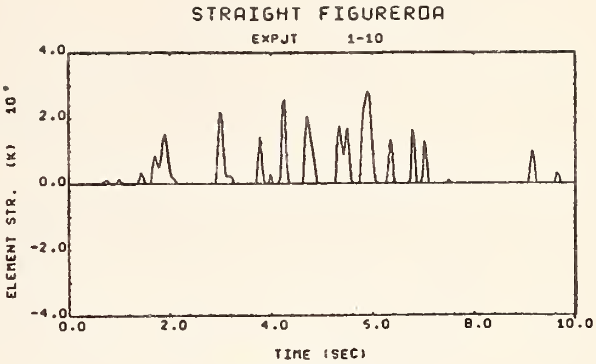


At Contact Point B

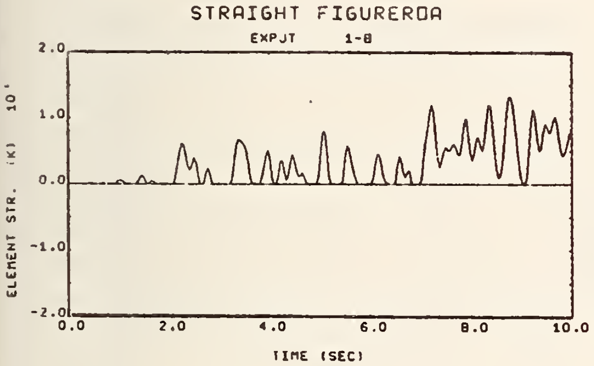
Fig. 101 Longitudinal Joint Separations at Expansion Joint # 1



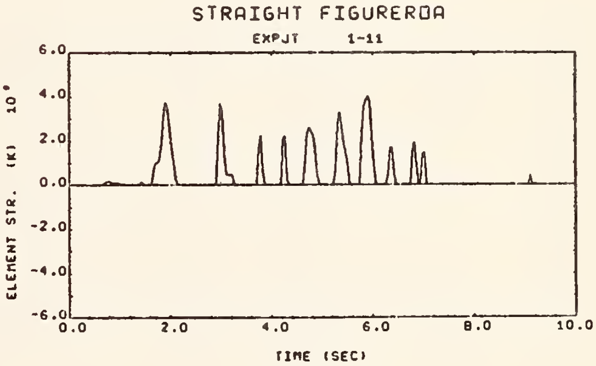
Tie Cable # 1



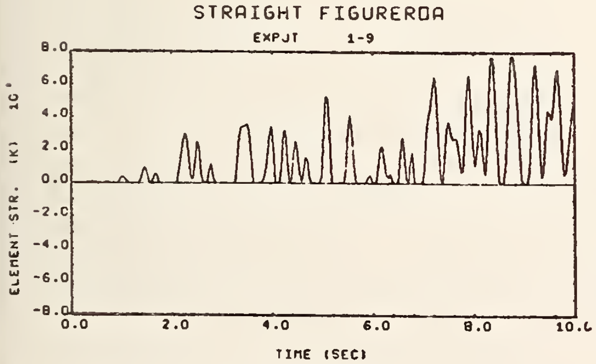
Tie Cable # 4



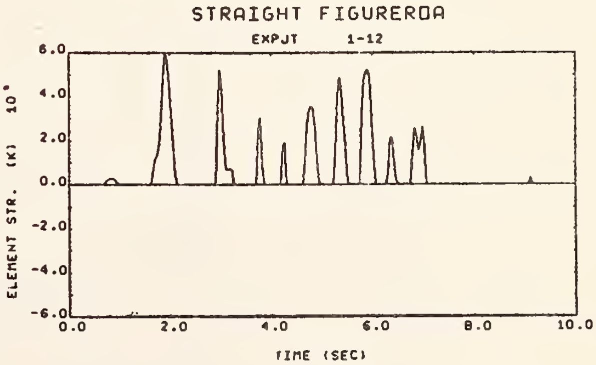
Tie Cable # 2



Tie Cable # 5



Tie Cable # 3



Tie Cable # 6

Fig. 102 Longitudinal Tie Cable Forces at Expansion Joint # 1

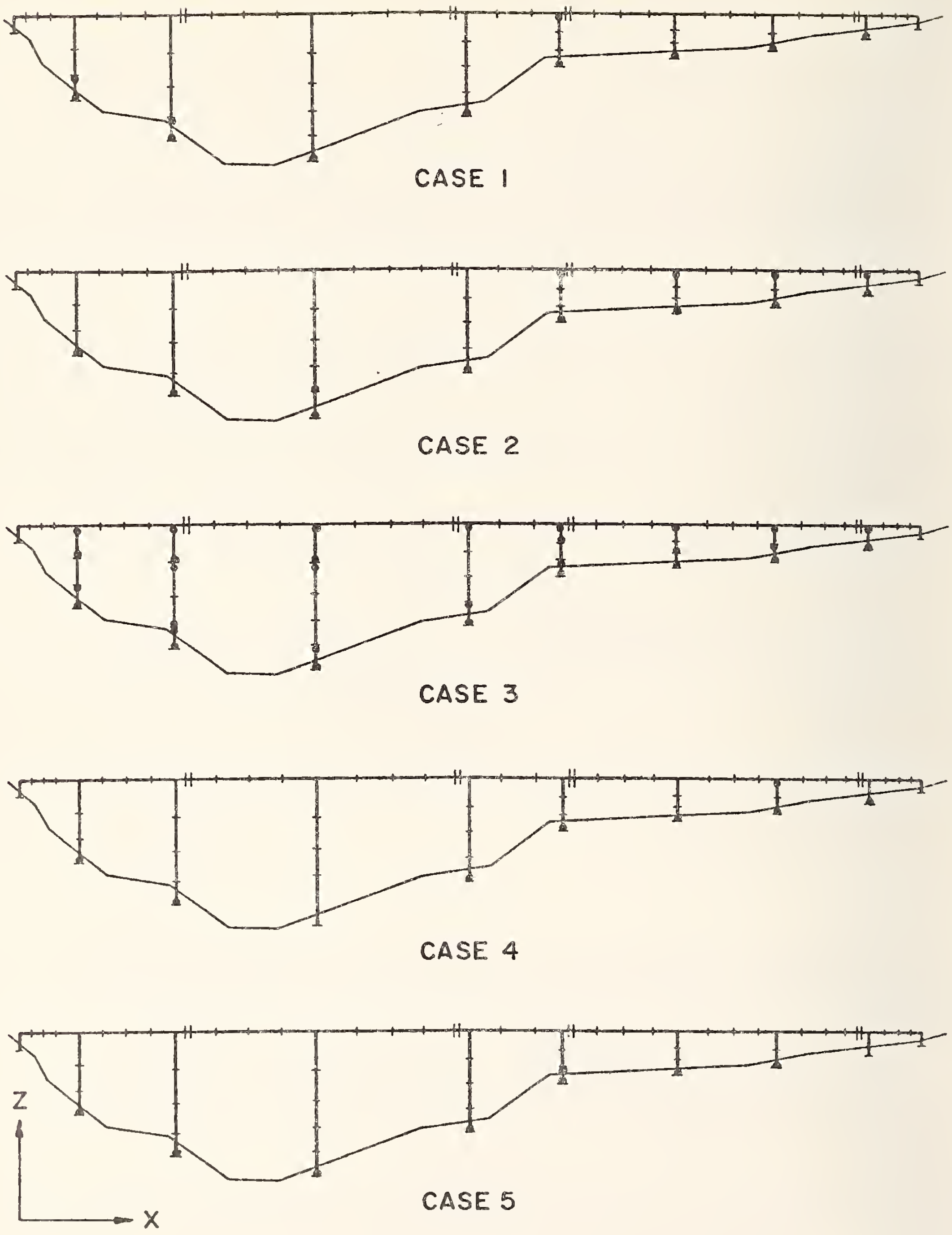


Fig. 103 Location of Maximum Flexural Yielding in Columns of 5/14 South Connector Overcrossing

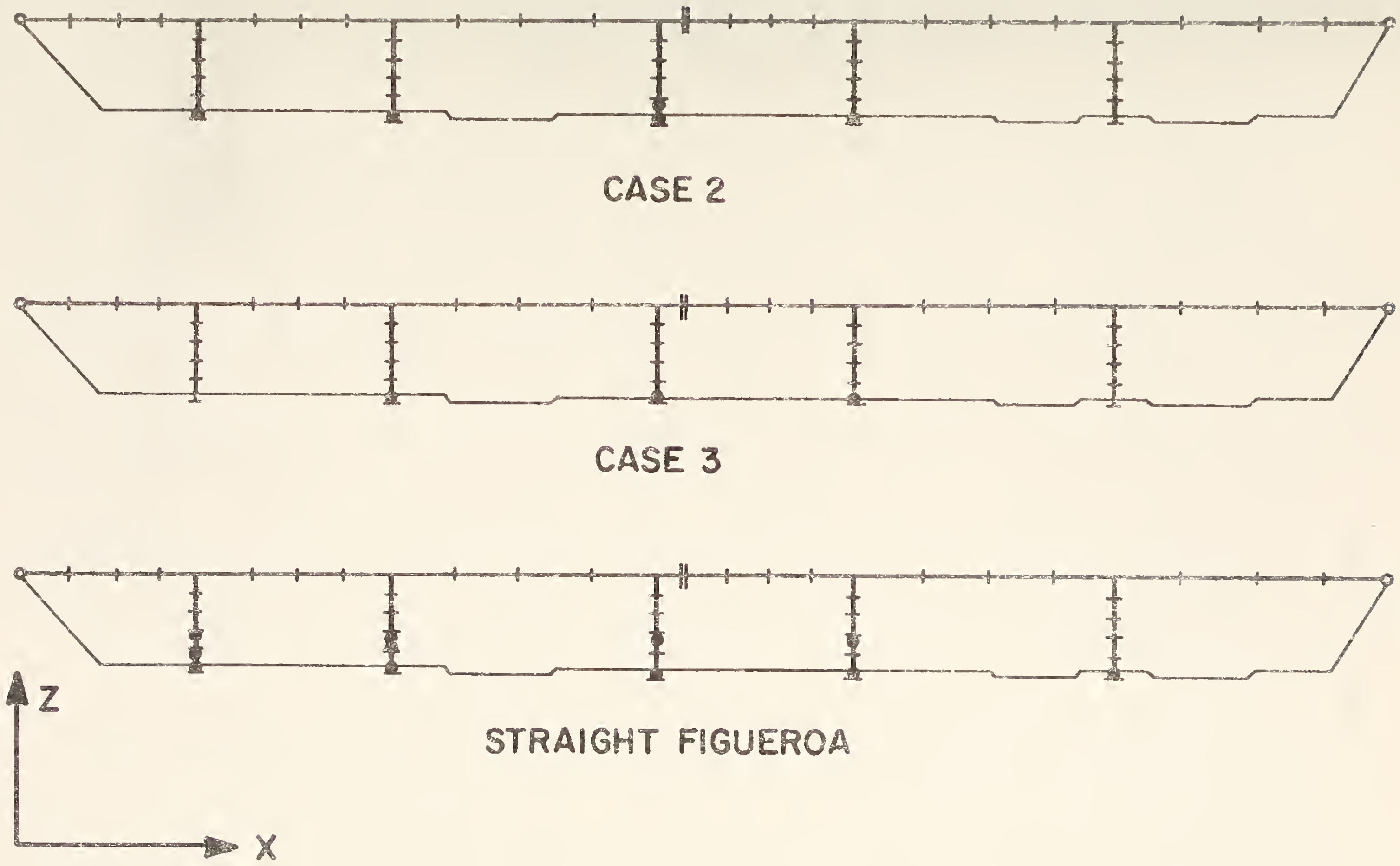


Fig. 104 Location of Maximum Flexural Yielding in Columns of the Curved and Straight Figueroa Street Undercrossing Connector

A. SEISMIC RESPONE CHARACTERISTICS OF BRIDGE SYSTEMS

The analytical results previously presented for the 5/14 South Connector Overcrossing showed that this bridge experienced large amplitude response when subjected to severe ground shaking (Cases 1, 2, 3, and 5). For Case 2 which uses weak short tie bars, the longitudinal deck separations exceeded the supporting ledge dimension (1.25 ft.) at expansion joints Nos. 1 and 2; see Table 27. The magnitudes of these separations correspond to tie bar ductility factors far in excess of those actually provided; thus, early failures of these bars would result allowing high amplitude response similar to Case 3 (no tie bars) to develop. This high amplitude response would result in collapse of the structure.

For Cases 1 and 5 which use strong short tie bars, the joint separations never exceeded the supporting ledge dimension even under conditions of severe shaking; thus, collapse of the structure in these 2 cases would not occur if the required tie ductilities are provided.

The analytical results presented for the curved Figueroa Street Undercrossing Connector (Case 2) showed relatively high amplitude acceleration response for the condition of severe excitations. Although this response causes some flexural damages at the bases of columns supporting the central spans, the oscillatory motions were quite stable. Due to the fact that the columns are relatively short and only one expansion joint is present in the deck, the deck displacements and the expansion joint separations are relatively small. The longitudinal restrainer cables at the joint never reach their yield condition (0.64 ft. elongation) even under severe shaking. The maximum joint separation in this case was 0.3 ft.

The horizontal time histories at the tops of all columns for

all bridge structures examined show that all segments of deck between expansion joints move essentially as rigid bodies in the horizontal directions, i.e, transverse bending effects in the deck are small. Therefore, the deck segments vibrate horizontally almost as rigid bodies linked together at the expansion joints. The principal mode of vibration resulting from transverse excitation is similar to the transverse fundamental mode for the linear zero friction case, i.e., similar to mode No. 2 shown in Fig. 17 for the 5/14 South Connector Overcrossing and mode No. 1 shown in Figs. 68 and 94 for the curved and straight Figueroa Street Undercrossing Connector, respectively.

It should be noted that for curved bridges, the transverse fundamental mode of vibration caused large forces to develop in the restrainer tie bars. Therefore, the properties of the restrainer tie bars greatly affect the transverse seismic responses of such bridges. These effects can be observed by comparing the results for Case 1 and 2 of the 5/14 South Connector Overcrossing.

The overall horizontal displacement responses for all cases of the 5/14 South Connector Overcrossing indicate drifts of the displacements in directions away from the centers of curvature of the decks, i.e., outward motions. This characteristic behavior is due to (1) the relatively high stiffnesses which develop as expansion joints close with inward displacements causing the deck to act as a continuous stiff arch from abutment to abutment, and (2) the relatively low stiffnesses which remain with outward displacements. For outward displacements, the stiffnesses are contributed only by the actions of columns and restrainer tie bars.

The drifts of horizontal displacements in the outward directions are particularly noticeable for those cases of loading which cause plastic deformations to occur in the columns and in the restrainer tie bars. Because of this biased behavior, the plastic deformations tend to accumulate in the outward direction. As a result, the entire bridge structure acts as a "shake-down" system.

In all cases of the curved Figueroa Street Undercrossing Connector, the overall horizontal displacement response was oscillatory and approximately equal in both inward and outward directions. The plastic

flexural rotation at the column bases was also oscillatory and did not accumulate in the outward direction as in the case of 5/14 South Connector Overcrossing. The stable response of the curved Figueroa Street Undercrossing is due to the fact that arch action of the deck does not occur because the amplitude of inward motions is insufficient to cause closure of the large (6 in.) joint gap in this bridge as indicated in Figs. 74, 82 and 89.

From the above comparison, it appears that for curved bridges, a bigger joint gap is desirable in order to avoid biased seismic responses and to prevent the shake-down situation under severe shaking conditions.

B. FLEXURAL YIELDING OF BRIDGE COLUMNS

For both 5/14 South Connector Overcrossing and Figueroa Street Undercrossing Connector, the location of the maximum flexural yielding in the columns (Figs 103 and 104) showed that under moderate shaking conditions flexural yielding was more likely to occur at the base of the columns (Case 4 of Fig. 103 and Case 3 of Fig. 104). Under severe shaking conditions, flexural yield hinges also developed at the top of the prismatic columns (Cases 1, 2, 3, and 5 of Fig. 103 and Cases 1 and 2 of Fig. 104). Columns located near expansion joints were shown to be more likely to develop yield hinges because of the relatively higher lateral displacement resulting from the fundamental transverse mode of vibration.

The damages due to flexural yielding at the top of a column will penetrate the deck and hence should be prevented. An effective method of achieving this is by flaring the upper portion of the column as in the case of Figueroa Street Undercrossing Connector.

The maximum local bending ductility factors at the base of some columns under severe shaking substantially exceeded the ductility that could actually be provided, however, under moderate shaking they remained within tolerable level (<5) which could be accommodated by well-designed columns. In all cases, the ductility factors required

by shorter columns are higher than for the longer columns. This results from the fact that deck segments vibrate laterally almost as rigid bodies causing similar lateral displacements of all columns and hence higher ductility requirements of the shorter columns.

The results of Case 1 (elastic columns) and Case 2 (elasto-plastic columns) of the curved Figueroa Street Undercrossing Connector indicate that elastic columns substantially increase the acceleration response in the deck without significantly affecting the displacement responses of the system.

C. EXPANSION JOINT RESTRAINERS

The results of Cases 1, 2, and 3 for 5/14 South Connector Overcrossing illustrate the effectiveness of longitudinal restrainers in reducing the seismic response of the bridge. It is clear from these results that longitudinal restrainers at expansion joints are essential in curved bridges with several expansion joints. The results of Case 2 illustrates the analytical inadequacy of the weak short tie bars, a fact exemplified by the bridge during the San Fernando earthquake. Cases 1 and 5 with strong short tie bars indicate sufficient strengths and stiffnesses but inadequate ductilities of these bars. The results of Table 27 showed that the required tie ductility factors of the strong short ties greatly exceeded the levels that these ties can accommodate even under moderate shaking (Case 4 in Table 27). Hence short tie bars are unsuitable for use as longitudinal restrainers at the expansion joints of long curved bridges because of insufficient reserved ductility.

The long cable longitudinal restrainers used at the expansion joint of Figueroa Street Undercrossing Connector had both sufficient strength and ductility to resist severe shaking conditions (Cases 1, 2 and straight). In all cases, the results showed that these cables did not experience yielding indicating that the length of the cables is more than required for resisting severe shaking. A reduction of 2 in length would have still enabled the cables to resist severe

shaking elastically.

The required length L_T of a longitudinal restrainer tie can be estimated from the expansion joint ledge dimension Δ_S and the yielding strain ε_T^Y of the tie by the relation

$$L_T = \frac{\Delta_S - \Delta_T}{\mu_T \ \varepsilon_T^Y} \tag{116}$$

where Δ_T is the tie gap and μ_T is the tie ductility factor (Eq. 115) which is a function of its material and size. Δ_S can be estimated from the maximum horizontal displacement response of the bridge and the geometry of the transverse fundamental mode of vibration for the linear zero friction case.

The purpose of using vertical restrainers at expansion joints is to prevent spans lifting from their supports. An examination of the time history response of the forces in the vertical restrainers at expansion joints Nos.1 and 2 of Cases 1 and 2 of 5/14 South Connector Overcrossing as shown in Figs. 34 and 45, indicates that under severe shaking tensile forces occur, although infrequently. The possibility of tensile forces in the vertical restrainers indicates the desirability of the use of the restrainers to prevent vertical separation of spans at the expansion joint.

D. INFLUENCE OF DECK CURVATURE

The influence of deck curvature on the dynamic characteristics of bridges was illustrated by the mode shapes of vibration shown in Figs. 68 and 93 for the curved and straight Figueroa Street Undercrossing Connector. It is clear from the results presented that horizontal curvature of the deck causes coupling between the two horizontal components, and superelevation of the deck causes coupling between vertical and horizontal components.

The effect of deck curvature on the seismic response of the

bridges was shown in the time history responses presented for the straight and Case 2 of the curved Figueroa Street Undercrossing Connector (Figs 84-88 and 76-81). Ground excitation in the transverse direction causes greater response in the straight bridge which in part is attributable to the greater lateral stability of the curved bridge resulting from the curved distribution of supporting columns and abutments.

E. INFLUENCE OF VERTICAL GROUND ACCELERATIONS

Vertical ground accelerations have an important effect on the vertical response of all bridges. They also significantly affect the horizontal response of curved bridges (Cases 1 and 5 of 5/14 South Connector Overcrossing) because of the coupling between the horizontal and vertical modes of vibration.

Vertical ground accelerations cause large vertical oscillatory motions in the deck resulting in a removal of the vertical compressive reaction at the expansion joints. This in turn removes the horizontal Coulomb friction force which affects the horizontal resistance of the structure. The vertical accelerations also change the axial load in the columns and therefore affect the ultimate bending strength of the columns due to yielding interaction effect.

F. INFLUENCE OF COULOMB FRICTION AT EXPANSION JOINTS

The results of modal analyses presented for 5/14 South Connector Overcrossing (Figs. 17 and 18) clearly showed that infinite friction at the expansion joints substantially increased the lateral stiffnesses of the bridge when compared to the case with zero friction. The case of zero friction is considered to be more practical and realistic, because the Coulomb friction force corresponding to the friction coefficient normally existing at expansion joints is relatively small when compared to the axial force which would result from the arch action of the deck in the infinite friction case.

Because Coulomb friction force is relatively small and acts only at a limited number of expansion joints, its influence on the total bridge response is small.

G. INFLUENCE OF OVERALL STRUCTURAL ARRANGEMENT

Although longitudinal restrainer ties are effective in reducing the amplitude of seismic response, they do not affect the dynamic characteristics of the bridge. This was illustrated by the essential similarity between the shapes of time history response of Case 1 and 2 of 5/14 South Connector Overcrossing. Longitudinal restrainers are secondary structural elements which are effective only when the primary horizontal seismic response is sufficiently large to activate them.

From the results presented for 5/14 South Connector Overcrossing and the curved Figueroa Undercrossing Connector, it is concluded that the major influence on the seismic response characteristics is the arrangement of supporting columns of each deck segment. For sufficient earthquake resistance of a bridge structure, each deck segment should be supported by at least 2 columns so that its lateral stiffness is reasonably adequate, and that stable oscillatory response of the structure can be achieved.

IX CONCLUSIONS AND RECOMMENDATIONS

A. CONCLUSIONS

Based on the analytical results presented for the bridge structures studied in this investigation, the following conclusions may be deduced:

(1) The basic mathematical models and the analytical procedures as well as computer programs developed in this investigation provide a rational and effective means for determining the seismic response of curved and straight, multiple-span, long, highway overcrossings of the type considered herein.

(2) The collapse of 5/14 South Connector Overcrossing during the San Fernando earthquake was due to the inability of the longitudinal restrainer ties to constrain the joint separation at expansion joints Nos. 1 and 2 thus causing the central spans between these expansion joints to fall off their supports resulting in the collapse of the spans along with column No. 4.

(3) The proposed curved Figueroa Street Undercrossing Connector can safely withstand severe ground shaking of an intensity similar to those experienced during the San Fernando earthquake; however, some flexural damage is likely to occur at the bases of the columns supporting the central spans.

(4) The strength and distribution of bridge columns in each deck segment between expansion joints are the primary factors which most significantly affect the seismic response characteristics of the bridge structure. Thus expansion joints in the deck should be located in a manner such that each substructural segment posses a reasonably adequate amount of lateral stiffnesses.

(5) Under severe shaking conditions similar to those experienced

during the San Fernando earthquake, the maximum horizontal acceleration response at the deck level substantially exceeded those values specified by the "Bridge Planning and Design Manual", California State Division of Highways [5] which was in effect prior to the San Fernando earthquake of February 9, 1971.

(6) Because of their effectiveness in reducing the amplitude of the seismic displacement response of curved bridges, longitudinal restrainer ties should be provided at expansion joints. They should have sufficiently high strength and ductility to constrain the joint separation within the limit of the supporting ledge dimension. Vertical restrainers should also be provided at expansion joints in order to prevent the lifting of spans from their supports.

(7) For curved bridges, a bigger expansion joint gap appears to be desirable to prevent biased seismic response and to avoid a shake-down situation under severe shaking conditions.

(8) The required expansion joint ledge dimension and joint gap can be estimated from the maximum horizontal displacement obtained from a seismic analysis and the geometry of the transverse fundamental mode of vibration.

(9) Under moderate to severe shaking, the overall seismic response of a bridge structure is considerably affected by vertical ground accelerations. The effect is particularly noticeable for curved bridges in which coupling exists between vertical and horizontal modes of vibration. Thus the vertical component of ground acceleration should always be included in a seismic analysis.

(10) For a normally proportioned bridge structure under moderate to severe shaking, flexural yielding is more likely to occur at the base of the columns. Yielding in columns should be avoided under moderate shaking but can be permitted to

limited levels under severe shaking conditions. It is therefore important that column reinforcement should be sized and detailed in such a manner that the columns can develop full yield strengths and maintain their yield capacities throughout their specified ranges of ductility. Flexural yielding should be prevented from developing at the top of columns in order to avoid damage penetrating the deck.

(11) The effect of Coulomb friction forces on the seismic response of the type of bridges considered is small, for the friction coefficient values normally existing at the expansion joints. Thus for linear analysis, a mathematical model assuming zero friction at expansion joints can be used as a realistic linear bridge model.

(12) Linear seismic response analysis provides a reasonable estimate of the maximum displacement response of a bridge system; however, substantial error may result in predicting the internal forces in the structure. Thus it is essential that nonlinear seismic analysis be carried out for major bridge structures under severe shaking conditions.

B. RECOMMENDATIONS

Due to the unique dynamic characteristics of bridge structures considered in this investigation, it is recommended that major highway overcrossings should be designed on the basis of dynamic analyses using appropriate nonlinear mathematical models to represent the structural systems. The dynamic analysis should include both vertical and horizontal components of ground motions suitable to the respective site. In determining the seismic response of long bridges, the "out-of-phasing" of the multiple support input ground motions may, in some cases, be significant. Therefore, every effort should be made to measure spatial correlations as well as time correlations for strong ground motions.

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APPENDIX I

FORTRAN IV LISTING OF PROGRAM YIELD

```
      PROGRAM YIELD (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT)
C
C
C     PROGRAM TO COMPUTE THE ULTIMATE AXIAL FORCE AND BIAXIAL BENDING
C     MOMENT FOR REINFORCED CONCRETE COLUMN OF ARBITRARY CROSS-SECTION
C
C
      COMMON/INPAR/ FC,E,EO,SY,TOLP,TOLE,NITRN
      DIMENSION T(3),HEAD(12)
C
      COMMON A(3000)
      MTOT=3000
C
   10 CALL SECOND(T(1))
C
C     INPUT PROGRAM CONTROL DATA AND MATERIAL PROPERTIES
C
      READ (5,1000) HEAD,NLOOP,NNODE,NBAR,NAS,NP,NE
      IF (NLOOP.EQ.0) STOP
      WRITE(6,2000) HEAD,NLOOP,NNODE,NBAR,NAS,NP,NE
C
      NITRN=30
      TOLP=1.0E-06
      TOLE=1.0E-04
C
      READ (5,1010) FC,E,EO,SY
      WRITE(6,2010) FC,E,EO,SY
C
C     COMPUTE ULTIMATE LOAD CAPACITIES
C
      N1=1
      N2=N1+NLOOP
      N3=N2+NP
      N4=N3+NE
      N5=N4+NE
      N6=N5+NAS
C
      NPE=NP*NE
      N61=N6+NPE
      N62=N61+NPE
      N63=N62+NPE
      N64=N63+NPE
      N65=N64+NPE
      N66=N65+NPE
      N67=N66+NPE
      N68=N67+NPE
C
      MNODE=NNODE+NLOOP
      N7=N68+MNODE
      N8=N7+MNODE
      N9=N8+MNODE
      N9=N8+MNODE
      N10=N9+MNODE
      N11=N10+MNODE
      N12=N11+MNODE
C
      N13=N12+NBAR
      N14=N13+NBAR
      N15=N14+NBAR
      N16=N15+NBAR
      N17=N16+NBAR*3
C
      NLOOP3=NLOOP*3
      N18=N17+NLOOP3
      N19=N18+NLOOP3
      N20=N19+NLOOP3
      N21=N20+NLOOP3
C
      N22=N21+NLOOP3
      N23=N22+NLOOP3
      N24=N23+NLOOP3
      N25=N24+NLOOP3
      N26=N25+NLOOP3
      N27=N26+NLOOP3
      N28=N27+NLOOP3
      N29=N28+NLOOP3
      N30=N29+NLOOP3
      IF (N30.LE.MTOT) GO TO 100
      WRITE(6,2020) N30
      STOP
C
  100 CALL ULOAD (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N61),
     1              A(N62),A(N63),A(N64),A(N65),A(N66),A(N67),
     2              A(N68),A(N7),A(N8),A(N9),A(N10),A(N11),A(N12),
     3              A(N13),A(N14),A(N15),A(N16),A(N17),A(N18),A(N19),
     4              A(N20),A(N21),A(N22),A(N23),A(N24),A(N25),A(N26),
     5              A(N27),A(N28),A(N29),NLOOP,NP,NE,NAS,MNODE,NNODE,
     6              NBAR )
C
      CALL SECOND (T(2))
      T(3)=T(2)-T(1)
      WRITE(6,2030) T(3)
      GO TO 10
C
 1000 FORMAT (12A6/6I5)
 1010 FORMAT (4F10.0)
C
 2000 FORMAT (1H1,12A6////
     *  28H NUMBER OF BOUNDARY LOOPS   = ,I5//
     *  28H NUMBER OF TOTAL NODES      = ,I5//
     *  28H NUMBER OF STEEL BARS       = ,I5//
     *  28H NUMBER OF STEEL AREAS      = ,I5//
     *  28H NUMBER OF AXIAL LOAD STEPS= ,I5//
     *  28H NUMBER OF MOMENT RATIOS    = ,I5///)
 2010 FORMAT (1H0,26HMATERIAL PROPERTIES OF THE SECTION..///
     *  39H ULTIMATE CONCRETE COMPRESSIVE STRESS = ,F14.4//
     *  39H YOUNG*S MODULUS OF STEEL BARS        = ,F14.4//
     *  39H ULTIMATE CONCRETE COMPRESSIVE STRAIN = ,F14.4//
     *  39H YIELDING STRESS OF STEEL BARS        = ,F14.4)
 2020 FORMAT (1H1,23HSTORAGE EXCEEDED---NEED ,I6/
     *  21H EXECUTION TERMINATED )
 2030 FORMAT (////23H TOTAL EXECUTION TIME = ,F8.2)
      END
```

```
      SUBROUTINE ULOAD (LNODE,RP,RE,DI,AS,P,UMX,UMY,UM ,SETA,DIST,TOP,
     *                  TOE,X,Y,ID,AC,ACX,ACY,XS,YS,IAS,IOS,ES,XA,YA,XB,YB,
     *                  ACA,ACAX,ACAY,ACB,ACBX,ACBY,ACAB,ACABX,ACABY,
     *                  NLOOP,NP,NE,NAS,MNODE,NNODE,NBAR)
C
C     COMPUTE ULTIMATE LOAD CAPACITIES FOR SPECIFIED LOAD COMBINATIONS
C
      COMMON/INFAP/FC,E,EO,SY,TOLP,TOLE,NITRN
C
      DIMENSION LNODE(NLOOP),RP(NP),RE(NE),DI(NE),AS(NAS),P(NP,NE),
     * UMX(NP,NE),UMY(NP,NE),UM (NP,NE),SETA(NP,NE),DIST(NP,NE),
     * TOP(NP,NE),TOE(NP,NE),
     * X(MNODE),Y(MNODE),ID(MNODE),AC(MNODE),ACX(MNODE),ACY(MNODE),
     * XS(NBAR),YS(NBAR),IAS(NBAR),IDS(NBAR),ES(NBAR,3),
     * XA(NLOOP,3),YA(NLOOP,3),XB(NLOOP,3),YB(NLOOP,3),
     * ACA(NLOOP,3),ACAX(NLOOP,3),ACAY(NLOOP,3),ACB(NLOOP,3),
     * ACBX(NLOOP,3),ACBY(NLOOP,3),ACAB(NLOOP,3),ACABX(NLOOP,3),
     * ACABY(NLOOP,3)
C
      COMMON/VALLE/SRA,CRA,D,DP
      DIMENSION F(3),G(3),GX(3),GY(3),T(3)
C
C     INPUT CROSS-SECTIONAL DATA
C
      WRITE(6,2000)
      READ (5,1010) (LNODE(L),L=1,NLOOP)
      WRITE(6,2010) (L,LNODE(L),L=1,NLOOP)
C
      WRITE(6,2015)
      NI=1
      DO 100 L=1,NLOOP
      NF=NI+LNODE(L)-1
      WRITE(6,2016) L,LNODE(L)
      READ (5,1020) (X(I),Y(I),I=NI,NF)
      WRITE(6,2020) (I,X(I),Y(I),I=NI,NF)
      NF=NF+1
      X(NF)=X(NI)
      Y(NF)=Y(NI)
      NI=NF+1
  100 CONTINUE
C
      READ (5,1030) (AS(I),I=1,NAS)
      READ (5,1040) (XS(J),YS(J),IAS(J),J=1,NBAR)
      DO 150 J=2,NBAR
      J1=J-1
  150 IF (IAS(J).EQ.0) IAS(J)=IAS(J1)
C
      WRITE(6,2025)
      WRITE(6,2030) (I,AS(I),I=1,NAS)
      WRITE(6,2035)
      WRITE(6,2040) (J,XS(J),YS(J),IAS(J),J=1,NBAR)
C
C     INPUT LOAD POINTS TO BE CALCULATED
C
      READ (5,1050) (RP(I),I=1,NP)
      READ (5,1060) (RE(I),DI(I),I=1,NE)
      READ (5,1070) IPO,IMXO,IMYO
C
      L=8*NPANE
      DO 155 I=1,L
  155 P(I)=0.
C
      ACT=0.
      NI=1
      DO 200 L=1,NLOOP
      NF=NI+LNODE(L)-1
      DO 160 I=NI,NF
      I1=I+1
      CALL AREA (X(I),Y(I),X(I1),Y(I1),AC(I),ACX(I),ACY(I))
  160 ACT=ACT+AC(I)
      NF=NF+1
      AC(NF)=0.
      ACX(NF)=0.
      ACY(NF)=0.
      NI=NF+1
  200 CONTINUE
C
      AST=0.
      DO 250 J=1,NBAR
      JS=IAS(J)
  250 AST=AST+AS(JS)
C
      POT=-SY*AST
      FCP=0.85*FC
      POC=FCP*ACT-POT
      RATIO=AST/ACT
      FRATIO=POT/POC
      WRITE(6,2050) ACT,POC,POT,AST,RATIO,FRATIO
      RATIO =FRATIO+0.1
C
C     ITERATION FOR COMPUTING LOAD COMBINATIONS
C
      DO 900 IE=1,NE
      RA=RE(IE)
      RA=RA*3.14159265359/180.
      REI=RA
      D=DI(IE)
      DO 800 IP=1,NP
C
      KKK=0
      IF (RP(IP).LT.RATIO) RP(IP)=RATIO
      IF (RP(IP).GT.0.9) RP(IP)=0.9
      RPUO=RP(IP)
      IF (RE(IE)-45.0) 297,297,298
  297 IND=-1
      EXEYO=TAN (REI)
      GO TO 299
C
  298 IND= 1
      EXEYO=TAN (3.14159265359/2.0-REI)
C
  299 ITRN=0
C
  300 ITRN=ITRN+1
      CRA=COS(RA)
      SRA=SIN(RA)
C
C     ID ARRAYS FOR NODES AND STEEL BARS
C
      YPMIN= 10000.
      YPMAX=-10000.
      NI=1
      DO 340 L=1,NLOOP
```

```
      NF=NI+LNCDE(L)-1
      DO 330 I=NI,NF
      IO(I)=0
      YP=X(I)*SRA+Y(I)*CRA-D
      IF (YP.GT.0.) IO(I)=1
      IF (YP.LE.YPMAX) GO TO 310
      YPMAX=YP
      IMAX=I
  310 IF (YP.GE.YPMIN) GO TO 330
      YPMIN=YP
      IMIN=I
  330 CONTINUE
      NF=NF+1
      IO(NF)=IO(NI)
      NI=NF+1
  340 CONTINUE
      OMAX=YPMAX-YPMIN
      IF ((YPMAX.LE.0.).OR.(YPMIN.GE.0.)) GO TO 655
C
      YIELDY=ABS(SY*YPMAX/(EO*E))
      DO 350 J=1,NBAR
      IOS(J)=0
      ES(J,2)=0.
      ES(J,3)=0.
      XP=XS(J)*CRA-YS(J)*SRA
      YP=XS(J)*SRA+YS(J)*CRA-D
      IF (ABS(YP).GE.YIELDY) GO TO 345
      FS(J,2)=-EO*(YPMAX-YP)/(YPMAX*YPMAX)
      ES(J,3)= EO*XP/YPMAX
  345 IF (YP.GE.YIELDY) IOS(J)=1
      IF (YP.LE.-YIELDY) IOS(J)=-1
      ES(J,1)= EO*YP/YPMAX
  350 CONTINUE
C
C     LOCATE POSITION OF NEUTRAL AXIS
C
      NLOOP39=NLCOP*39
      DP=0.15*YPMAX
      DO 355 I=1,NLCOP39
  355 XA(I)=0.
C
      NI=1
      DO 390 L=1,NLCOP
      NF=NI+LNCDE(L)-1
      DO 380 I=NI,NF
      I1=I+1
      IF (IO(I)-IO(I1)) 360,380,370
C
  360 CALL NEUTRAL (XA(L,1),YA(L,1),XA(L,2),YA(L,2),XA(L,3),YA(L,3),
     *                 X(I),Y(I),X(I1),Y(I1))
C
      CALL AREA (XA(L,1),YA(L,1),X(I1),Y(I1),ACA(L,1),ACAX(L,1),
     *           ACAY(L,1))
C
      CALL DIFFRNT (ACA(L,2),ACAX(L,2),ACAY(L,2),
     *              ACA(L,3),ACAX(L,3),ACAY(L,3),
     *              XA(L,1),YA(L,1),XA(L,2),YA(L,2),XA(L,3),YA(L,3),
     *              X(I1),Y(I1),0.0,0.0,0.0,0.0)
      GO TO 380
C
  370 CALL NEUTRAL (XB(L,1),YB(L,1),XB(L,2),YB(L,2),XB(L,3),YB(L,3),
     *                 X(I),Y(I),X(I1),Y(I1))
C
      CALL AREA (X(I),Y(I),XB(L,1),YB(L,1),ACB(L,1),ACBX(L,1),ACBY(L,1))
C
      CALL DIFFRNT (ACB(L,2),ACBX(L,2),ACBY(L,2),
     *              ACB(L,3),ACBX(L,3),ACBY(L,3),
     *              X(I ),Y(I ),0.0,0.0,0.0,0.0,
     *              XB(L,1),YB(L,1),XB(L,2),YB(L,2),XB(L,3),YB(L,3))
  380 CONTINUE
      NF=NF+1
      NI=NF+1
C
      CALL AREA (XB(L,1),YB(L,1),XA(L,1),YA(L,1),
     *           ACAB(L,1),ACABX(L,1),ACABY(L,1))
C
      CALL DIFFRNT (ACAB(L,2),ACABX(L,2),ACABY(L,2),
     *              ACAB(L,3),ACABX(L,3),ACABY(L,3),
     *              XB(L,1),YB(L,1),XB(L,2),YB(L,2),XB(L,3),YB(L,3),
     *              XA(L,1),YA(L,1),XA(L,2),YA(L,2),XA(L,3),YA(L,3))
C
  390 CONTINUE
C
C     COMPUTE ULTIMATE LOADS
C
      PU=0.0
      PUX=0.0
      PUY=0.0
      NI=1
      DO 450 L=1,NLCOP
      NF=NI+LNCDE(L)-1
      DO 440 I=NI,NF
      I1=I+1
      IF ((IO(I).LT.1).OR.(IO(I1).LT.1)) GO TO 440
      PU=PU+AC(I)
      PLX=PLX+ACY(I)
      PUY=PLY+ACX(I)
  440 CONTINUE
      NF=NF+1
      NI=NF+1
      PU=PU+ACA(L,1)+ACB(L,1)+ACAB(L,1)
      PUX=PLX+ACAY(L,1)+ACBY(L,1)+ACABY(L,1)
      PUY=PLY+ACAX(L,1)+ACBX(L,1)+ACABX(L,1)
  450 CONTINUE
      PU=FCP*PU
      PUX=FCP*PLX
      PUY=FCP*PUY
C
      DO 500 J=1,NBAR
      JS=IAS(J)
      ASJ=AS(JS)
      IF (IOS(J)) 460,470,480
  460 FSJ=-SY*ASJ
      GO TO 490
  470 FSJ=E*ES(J,1)*ASJ
      GO TO 490
  480 FSJ=SY*ASJ
  490 PU=PU+FSJ
      PLX=PLX+FSJ*YS(J)
      PUY=PLY+FSJ*XS(J)
  500 CONTINUE
C
C     CHECK CONVERGENCE
C
```

```
      RPU=PL/POC
  520 RORE=0.
      IF (IND) 525,525,527
  525 EXEY=PUY/PUX
      THETA=ATAN(EXEY)*180./3.14159265359
      GO TO 532
C
  527 EXEY=PUX/PUY
      THETA=ATAN (EXEY) *180./3.14159265359
      THETA=90.-THETA
  532 RORE=ABS (EXEY-EXEYO)
      RDRP=ABS (RPL-RPUO)
      ANGLE=RA*180./3.14159265359
C
      IF ((RORE.LE.TOLE).AND.(RDRP.LE.TOLP)) GO TO 700
C
C     COMPUTE NEW LOCATION OF NEUTRAL AXIS
C
      DO 550 I=1,3
      F(I)=0.
      G(I)=0.
      GX(I)=0.
      GY(I)=0.
  550 CONTINUE
      DO 600 L=1,NLCOP
      DO 600 I=2,3
      F(I)=F(I)+ACA(L,I)+ACB(L,I)+ACAB(L,I)
      GX(I)=GX(I)+ACAY(L,I)+ACBY(L,I)+ACABY(L,I)
      GY(I)=GY(I)+ACAX(L,I)+ACBX(L,I)+ACABX(L,I)
  600 CONTINUE
      DO 610 I=2,3
      F(I)=FCP*F(I)
      GX(I)=FCP*GX(I)
      GY(I)=FCP*GY(I)
  610 CONTINUE
C
      DO 620 J=1,NBAR
      IF (IC(J).NE.0) GO TO 620
      JS=IAS(J)
      ASJ=AS(JS)
      FSDJ=E*ES(L,2)*ASJ
      FSRAJ=E*ES(J,3)*ASJ
      F(2)=F(2)+FSDJ
      F(3)=F(3)+FSRAJ
      GX(2)=GX(2)+FSDJ*YS(J)
      GY(2)=GY(2)+FSDJ*XS(J)
      GX(3)=GX(3)+FSRAJ*YS(J)
      GY(3)=GY(3)+FSRAJ*XS(J)
  620 CONTINUE
      F(2)=F(2)/POC
      F(3)=F(3)/POC
C
  630 ORA=0.
      ORP=RP(IP)-RPC
      AREI=ABS(FE(IE))
      IF ((AREI.LT.1.0E-10).OR.(AREI.GE.90.)) GO TO 650
      DRE=EXEYO-EXEY
      IF (IND) 635,635,640
  635 G(2)=(GY(2)*PLX-GX(2)*PUY)/(PUX*PUX)
      G(3)=(GY(3)*PLX-GX(3)*PUY)/(PUX*PUX)
      GO TO 645
  640 G(2)=(GX(2)*PUY-GY(2)*PUX)/(PUY*PUY)
      G(3)=(GX(3)*PLY-GY(3)*PUX)/(PUY*PUY)
C
  645 FG=F(2)*G(3)-F(3)*G(2)
      IF (ABS(FG).LT.1.0E-10) GO TO 650
      OD=(ORP*G(3)-DRE*F(3))/FG
      ORA=(-DRP*G(2)+DRE*F(2))/FG
      OD=OD/2.0
      DRA=DRA/2.0
      GO TO 690
  650 IF (ABS(F(2)).GT.1.0E-15) GO TO 660
  655 KKK=1
      GO TO 700
  660 OD=DRP/F(2)
      DD=DD/2.0
  690 OMAY=CMAX/4.C
      IF (CC.GE.OMAY) DD=DMAX/5.
      IF (DD.LE.-DMAY) DD=-DMAX/5.
      D=D+DD
      AORA=ABS(DRA)
      IF (ACRA.GE.C.1) DRA=DRA*0.1/AORA
      RA=RA+DRA
C
  699 IF (ITRN.LT.NITRN) GO TO 300
C
C     OUTPUT RESULTS
C
  700 CONTINUE
      UM(IP,IE)=SQRT(PUX*PUX+PUY*PUY)
      SETA(IP,IE)=ANGLE
      DIST(IP,IE)=C
      TCP(IP,IE)=RCRP
      TOE(IP,IE)=PCRE
      P(IP,IE)=PU
      UMX(IP,IE)=PLX
      UMY(IP,IE)=PLY
      IF (KKK.LT.1) GO TO 800
      D=0.
      RA=REI
  800 CONTINUE
  900 CONTINUE
C
      WRITE(6,2300)
      DO 950 J=1,NE
      DO 940 I=1,NP
      IF (I.EQ.1) GO TO 930
      WRITE(6,2400) RP(I)  ,P(I,J),UMX(I,J),UMY(I,J),UM (I,J),SETA(I,J)
     *                DIST(I,J),TOP(I,J),TCE(I,J)
      GO TO 940
  930 WRITE(6,2500) RE(J)  ,RP(I)  ,P(I,J),UMX(I,J),UMY(I,J),
     *                UM (I,J),SETA(I,J),DIST(I,J),TCP(I,J),TCE(I,J)
  940 CONTINUE
  950 CONTINUE
C
      I=IPO*IMXO*IMYO
      IF (I.EQ.0) GO TO 1000
      WRITE(6,2600)
      DO 990 J=1,NE
      UMO=UM(IPC,J)
      DO 980 I=1,NP
      SETA(I,J)=UMX(I,J)/UM(I,IMXO)
      DIST(I,J)=UMY(I,J)/UM(I,IMYO)
      TOP(I,J)=LP(I,J)/UMO
```

```
      IF (I.EQ.1) GO TO 970
      WRITE(6,2700)      RP(I),P(I,J),UMX(I,J),UMY(I,J),UM(I,J),
     *                   SETA(I,J),DIST(I,J),TOP(I,J)
      GO TO 980
  970 WRITE(6,2800) RE(J),RP(I),P(I,J),UMX(I,J),UMY(I,J),UM(I,J),
     *                   SETA(I,J),DIST(I,J),TOP(I,J)
  980 CONTINUE
  990 CONTINUE
 1000 CONTINUE
      RETURN
C
 1010 FORMAT (4I5)
 1020 FORMAT (2F10.0)
 1030 FORMAT (8F10.0)
 1040 FORMAT (2F10.0,I5)
 1050 FORMAT (8F10.0)
 1060 FORMAT (2F10.0)
 1070 FORMAT (3I5)
 2000 FORMAT (1H1,30HCOLUMN CROSSSECTIONAL DATA....    /)
 2010 FORMAT (5H LOOP,10X,9HNUMBER OF  /
     *        5H  NO.,10X,9H  NODES     /(I5,I16))
 2015 FORMAT ( ///24HCOORDINATES OF NODES....    /)
 2016 FORMAT (10H LOOP NO. ,I3,5H  HAS,I5,8H   NODES  /)
 2020 FORMAT (5H NODE,10X,1HX,16X,1HY//(I5,2F16.5))
 2025 FORMAT (1H1,31HNUMBER OF DIFFERENT STEEL AREAS  // 5H  NO.,10X,
     *        4HAREA   /)
 2030 FORMAT (I5,F16.5)
 2035 FORMAT ( ///30H COORDINATES OF STEEL BARS....    //
     *        5H  BAR,10X,1HX,15X,1HY,15X,8HAREA NO.    /)
 2040 FORMAT (I5,2F16.5,I14)
 2050 FORMAT (1H1/ 20H REFERENCE DATA.....  ///
     *  27H TOTAL AREA OF THE SECTION=  ,F16.5//
     *  27H ULTIMATE COMPRESSIVE LOAD=  ,F16.5//
     *  27H ULTIMATE TENSILE LOAD    =  ,F16.5//
     *  27H TOTAL REINFORCEMENT AREA =  ,F16.5//
     *  27H REINFORCEMENT RATIO      =  ,F16.5//
     *  27H TENSILE LOAD RATIO       =  ,F16.5)
 2300 FORMAT (1H1,28HPRINT OF FINAL RESULTS......  /
     *  5X,2HRE, 8X,2HRP, 9X,2HPU,11X,3HMUX,10X,3HMUY,10X,3HUM ,10X,
     *  4HSETA, 9X,4HDIST,9X,3HTOP,10X,3HTOE  /)
 2400 FORMAT (1H ,10X,F8.3,2X,8E13.4 )
 2500 FORMAT (/1H ,F8.3,2X,F8.3,2X,8E13.4 )
 2600 FORMAT (1H1
     *  5X,2HPE, 8X,2HRP, 9X,2HPU,11X,3HMUX,10X,3HMUY,10X,3HUM ,10X,
     *  7HUMX/MXO,6X,7HUMY/MYO,6X,6HUM/UMO    /)
 2700 FORMAT (1H ,10X,F8.3,2X,4E13.4,3F13.4)
 2800 FORMAT (/1H ,F8.3,2X,F8.3,2X,4E13.4,3F13.4)
      END


      SUBROUTINE NEUTRAL (XA,YA,XAO,YAO,XARA,YARA,X,Y,XK,YK)
C
      COMMON/VALUE/SRA,CRA,D,OP
C
      XX=XK-X
      YY=YK-Y
      AXX=ABS(XX)
      AYY=ABS(YY)
      IF ((AXX.LT.1.0E-15).AND.(AYY.GT.1.0E-15)) GO TO 400
      IF ((AXX.GT.1.0E-15).AND.(AYY.LT.1.0E-15)) GO TO 500
      IF ((AXX.LT.1.0E-15).AND.(AYY.LT.1.0E-15)) GO TO 600
C
      OB=XX*SRA+YY*CRA
      IF (ABS(OB).GT.1.0E-15) GO TO 300
      STOP
C
  300 XA=X*YK-XK*Y
      YA=(-XA*SRA+(D+OP)*YY)/OB
      XA=(XA*CRA+(D+OP)*XX)/OB
      XAD=0.85*XX/CB
      YAO=0.85*YY/CB
      XARA=(-(X*YK-XK*Y)*SRA-XA*(-XX*CRA+YY*SRA))/OB
      YARA=(-(X*YK-XK*Y)*CRA-YA*(-XX*CRA+YY*SRA))/OB
      RETURN
C
  400 XA=X
      YA=(D+OP-X*SRA)/CRA
      XAO=0.
      YAO=0.85/CRA
      XARA=0.
      YARA=-(X (D+OP)*SRA)/(CRA*CRA)
      RETURN
C
  500 XA=(D+OP-Y*CRA)/SRA
      YA=Y
      XAD=0.85/SRA
      YAO=0.
      XARA=(Y-(D+OP)*CRA)/(SRA*SRA)
      YARA=0.
      RETURN
C
  600 XA=X
      YA=Y
      RETURN
      END


      SUBROUTINE AREA (X,Y,X1,Y1,A,AX,AY)
C
      XX=X-X1
      YY=Y+Y1
      A=XX*YY/2.0
      AX=XX*(X*(YY+Y)+X1*(YY+Y1))/6.0
      AY=XX*(YY*YY-Y*Y1)/6.0
      RETURN
      END


      SUBROUTINE DIFFRNT (AD,AXD,AYD,ARA,AXRA,AYRA,
     *                   XA,YA,XAD,YAO,XARA,YARA,
     *                   XB,YB,XBD,YBD,XBRA,YBRA)
C
      XX=XA-XB
      YY=YA+YB
      XXD=XAD-XBD
      YYD=YAO+YBD
      XXRA=XARA-XBRA
      YYRA=YARA+YBRA
```

```
ULOA 484  C
ULOA 485        AD=(XXD*YY+XX*YYD)/2.0
ULOA 486        ARA=(XXRA*YY+XX*YYRA)/2.0
ULOA 487        AXD=(XXD*(XA*(YA+YY)+XB*(YY+YB))+
ULOA 488       *        XX*(XAD*(YY+YA)+XBD*(YY+YB)+XA*(YYD+YAD)+XB*(YYD+YBD)))/6.0
ULOA 489        AXRA=(XXRA*(XA*(YA+YY)+XB*(YY+YB))+
ULOA 490       *   XX*(XARA*(YA+YY)+XBRA*(YY+YB)+XA*(YYRA+YARA)+XB*(YYRA+YBRA)))/6.
ULOA 491        AYD=(XXD*(YY*YY-YA*YB)+XX*(2.0*YY*YYD-YAD*YB-YA*YBD))/6.0
ULOA 492        AYRA=(XXRA*(YY*YY-YA*YB)+XX*(2.0*YY*YYRA-YARA*YB-YA*YBRA))/6.0
ULOA 493        RETURN
ULOA 494        END
```

APPENDIX II

FORTRAN IV LISTING OF PROGRAM BSAP

```
      PROGRAM BSAP (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT,TAPE1,TAPE2
     .              ,TAPE3,TAPE4,TAPE7,TAPE8,TAPE9,TAPE10,TAPE11,TAPE12)
C
C
C     BSAP....A MODIFIED VERSION OF ...SAP... PROGRAM
C             FOR STATIC AND DYNAMIC LINEAR ELASTIC ANALYSIS
C             OF 3-DIMENSIONAL BRIDGE STRUCTURAL SYSTEMS.
C
C
      COMMON/JUNK/ HED(12),JUK(193)
      COMMON/ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON/EM/ DCCO(1058)
      COMMON/MISC/ NBLOCK,NEQB,LL,NF,LB,NSVV,NFN,NGM,NAT,N7,NOT
C
      DIMENSION T(7)
C
C     PROGRAM CAPACITY CONTROLLED BY THE FOLLOWING TWO STATEMENTS...
C
      COMMON A(20000)
      MTOT=20000
C
C     PROGRAM CONTROL DATA
C
    5 CALL SECOND (T(1))
      READ (5,100) HED,NUMNP,NELTYP,NDYN
      IF (NUMNP.EQ.0) STOP
      WRITE(6,200) HED,NUMNP,NELTYP,NDYN
C
C     ANALYSIS TYPE CONTROL DATA
C
      READ (5,101) LL,NF,NSVV,NFN,NGM,NAT,N7,NOT
      IF (NSVV.LT.10) NSVV=10
      IF (NDYN.EQ.0) WRITE(6,201) LL
      IF (NDYN.GT.0) WRITE(6,202) NF,NSVV
      IF (NDYN.EQ.2) WRITE(6,203) NFN,NGM,NAT,N7,NOT
C
C     INPUT JOINT DATA----ID ARRAY STORED ON TAPE 8
C
      N1=1
      N2=N1+6*NUMNP
      N3=N2+NUMNP
      N4=N3+NUMNP
      N5=N4+NUMNP
      IF (N5.GT.MTOT) CALL ERROR(N5-MTOT)
      CALL INPUTM (A(N1),A(N2),A(N3),A(N4),NUMNP,NEQ,NEQ,NDYN,NGM)
C
C     FORM ELEMENT STIFFNESSES----STIFF. ON TAPE 2, STRESS ON TAPE 1
C
      CALL SECOND(T(2))
      CALL ELSTF  (NUMEL)
C
C     COMPUTE MAX. PERMISSIBLE NUMBER OF EQUATIONS IN A BLOCK
C
      CALL SECOND (T(3))
      CALL BLOCFC (NEQB,NL,NDYN)
      NBLOCK=(NEQ-1)/NEQB+1
      IF (NEQB.GT.NEQ) NEQB=NEQ
      WRITE(6,204) NEQ,MBAND,NEQB,NBLOCK
C
C     INPUT NODAL LOADS AND MASSES----STORED ON TAPE 3
C
      N3=N2+NEQB*LL
      N4=N3+6*LL
      CALL SECOND (T(4))
      CALL INL  (A(N1),A(N2),A(N3),A(N4),NUMNP,NEQB,LL,NF,NEQ)
C
C     FORM TOTAL STIFFNESS---ON TAPE 4, MASS MATRIX---ON TAPE 9
C
      NE2B=2*NEQB
      N2=N1+NEQB*MBAND
      N3=N2+NEQB*LL
      N4=N3+4*LL
      NN2=N1+NE2B*MBAND
      NN3=NN2+NE2B*LL
      NN4=NN3+4*LL
      CALL SECOND (T(5))
      CALL ADDSTF  (A(N1),A(NN2),A(NN3),A(NN4),NUMEL,NBLOCK,NE2B,LL,
     .              MBAND,NL,NEQ)
C
      CALL SECOND (T(6))
C
C     CALL FOR APPROPRIATE SOLUTION TYPE
C
      IF (NDYN.EQ.0) CALL STATIC
      IF (NDYN.EQ.1) CALL MODES
      IF (NDYN.EQ.2) CALL HISTRY
      IF (NDYN.EQ.3) CALL RESPEC
C
      CALL SECOND (T(7))
C
      DO 40 I=1,6
   40 T(I)=T(I+1)-T(I)
      T(7)=T(1)+T(2)+T(3)+T(4)+T(5)+T(6)
      WRITE(6,205) T
C
      GO TO 5
C
  100 FORMAT (12A6/3I5)
  101 FORMAT (8I5)
  200 FORMAT (1H1,12A6///
     .  28H NUMBER OF NODAL POINTS     =  ,I5//
     .  28H NUMBER OF ELEMENT TYPES    =  ,I5//
     .  28H ANALYSIS TYPE CODE NO.     =  ,I5//)
  201 FORMAT (28H  NUMBER OF LOAD CASES      =  ,I5//)
  202 FORMAT (28H  NUMBER OF FREQUENCIES     =  ,I5//
     .             28H  EIGEN-SUBSPACE PARAMETER =  ,I5//)
  203 FORMAT (28H  NUMBER OF TIME FUNCTIONS =  ,I5//
     .             28H  GROUND MOTION INPUT CODE =  ,I5//
     .             28H  NUMBER OF DELAYED TIMES  =  ,I5//
     .             28H  NUMBER OF TIME STEP DT   =  ,I5//
     .             28H  DT MULTIPLIER FOR OUTPUT =  ,I5//)
  204 FORMAT (34H1 TOTAL NUMBER OF EQUATIONS          =  ,I5,
     .           /34H  BANDWIDTH                        =  ,I5,
     .           /34H  NUMBER OF EQUATIONS IN A BLOCK   =  ,I5,
     .           /34H  TOTAL NUMBER OF BLOCKS           =  ,I5)
  205 FORMAT (/12H OVERALL LOG  //
     .              33H  NODAL POINT INPUT.............  ,F8.2//
     .              33H  FORM ELEMENT STIFFNESSES......  ,F8.2//
     .              33H  DETERMINE EQUATION BLOCK SIZE.  ,F8.2//
     .              33H  INPUT NODAL LOADS AND MASSES..  ,F8.2//
     .              33H  FORM TOTAL STIFFNESS..........  ,F8.2//
     .              33H  PARTICULAR SOLUTION (ABOVE)...  ,F8.2//
     .             /33H  TOTAL SOLUTION TIME...........  ,F8.2//)
```

```
      END
```

```
      SUBROUTINE ERROR(N)
      WRITE (6,2000) N
 2000 FORMAT (// 20H STORAGE EXCEEDED BY  I6)
      STOP
      END
```

```
      SUBROUTINE INPUTM(ID,X,Y,Z,NUMNP,NEQ,NEQG,NDYN,NGP)
C
C     NODAL POINT INPUT AND GENERATION.
C
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,6) ,NID(6)
      ND=6
      WRITE(6,200)
      WRITE (6,204)
      KC=1
      DO 70 I=1,6
   70 NID(I)=0
   11 READ (5,100) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N),KN
      WRITE(6,203) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N),KN
      DO 60 I=1,6
      IF (ID(N,I)) 61,62,63
   61 IF (ID(N,I).EQ.-2) GO TO 65
      NID(I)=-1
      ID(N,I)=1
      GO TO 60
   62 IF (NID(I).EQ.-1) ID(N,I)=1
      GO TO 60
   63 NID(I)=0
      GO TO 60
   65 ID(N,I)=-1
   60 CONTINUE
      IF(KC.EQ.1) GO TO 12
C
C     CHECK IF GENERATION NEEDED
C
      IF (KN) 10,10,20
   12 KC=0
   10 CONTINUE
      NUMINT=1
      GO TO 15
C
C     GENERATE NEW NODES
C
   20 NUMINT=(N-NI)/KN
      DX=(X(N)-X(NI))/NUMINT
      DY=(Y(N)-Y(NI))/NUMINT
      DZ=(Z(N)-Z(NI))/NUMINT
      DO 21 J=1,NUMINT
      NN=NI+J*KN
      X(NN)=X(NN-KN)+DX
      Y(NN)=Y(NN-KN)+DY
      Z(NN)=Z(NN-KN)+DZ
C
C     SET JOINT DOF CODE... SAME AS FIRST JOINT IN A SERIES
C
      DO 22 JJ=1,6
      IF(ID(NI,JJ)-1) 24,26,25
C
C     GENERATE NEW MASTER NODES
C
   25 ID(NN,JJ)=ID(NI,JJ)+J*KN
      GO TO 22
   26 ID(NN,JJ)=ID(NI,JJ)
      GO TO 22
   24 IF (ID(NI,JJ).NE.-2) GO TO 23
      ID(NN,JJ)=ID(NI,JJ)
      GO TO 22
```

```
   23 ID(NN,JJ)=0
   22 CONTINUE
   21 CONTINUE
   15 NI=N
C
C     CHECK FOR LAST NODAL POINT
C
      IF(NUMNP-NI) 13,13,11
   13 CONTINUE
C
C     PRINT ALL NODAL DATA
C
      WRITE (6,202)
      WRITE (6,204)
      DO 50 N=1,NUMNP
   50 WRITE(6,203) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N)
C
C      NUMBER UNKNOWNS AND SET MASTER NODE NEGATIVE.
C
      NEQ=0
      DO 40 I=1,NUMNP
      DO 40 J=1,ND
      IF (ID(I,J).LT.0) GO TO 40
      IF(ID(I,J)-1) 37,38,39
   37 NEQ=NEQ+1
      ID(I,J)=NEQ
      GO TO 40
   38 ID(I,J)=0
      GO TO 40
   39 ID(I,J)=-ID(I,J)
   40 CONTINUE
C
      NEQG=NEQ
      IF (NCYN.NE.2) GO TO 90
      IF (NCM.NE.2) GO TO 90
C
      DO 80 I=1,NUMNP
      DO 80 J=1,ND
      IF (ID(I,J).NE.-1)  GO TO 80
      NEQG=NEQG+1
      ID(I,J)=NEQG
   80 CONTINUE
C
      WRITE(6,205)
      WRITE(6,204)
      DO 89 N=1,NUMNP
   89 WRITE(6,203) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N)
C
   90 CONTINUE
      REWIND 8
      WRITE (8) ID
C
      RETURN
  100 FORMAT (7I5,3F10.0,I5)
  104 FORMAT (I15,4F10.4)
C
  200 FORMAT (26H1NODAL POINT DATA AS INPUT//)
  202 FORMAT (26H1COMPLETE NODAL POINT DATA  //   )
  203 FORMAT (I15,6I5,3F13.3,I5)
  204 FORMAT (5HONODE 6X 24HBOUNDARY CONDITION CODES 13X
     .  23HNODAL POINT COORDINATES / 7H NUMBER 2X 1HX 4X 1HY 4X 1HZ 3X
     . 2HXX 3X 2HYY 3X 2HZZ12X 1HX 12X 1HY 12X 1HZ/)
  205 FORMAT (26H1INTERNAL NODAL POINT DATA  // )
C
      END
```

```
      SUBROUTINE ELSTIF(NUMEL)
C
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
C
      MBAND=0
      NUMEL=0
      REWIND 1
      REWIND 2
      DO 900 M=1,NELTYP
C
      READ  (5,1001) NPAR
      WRITE (1) NPAR
      NUMEL=NUMEL+NPAR(2)
      MTYPE=NPAR(1)
C
      GO TO (1,2,3,4,5)  MTYPE
C
C     THREE DIMENSIONAL TRUSS ELEMENTS
C
    1 CALL TRUSS
      GO TO 900
C
C     THREE DIMENSIONAL BEAM ELEMENTS
C
    2 CALL BEAM
      GO TO 900
C
C     THREE DIMENSIONAL CURVED BEAM ELEMENTS
C
    3 CALL CBEAM
      GO TO 900
C
C     BOUNDARY ELEMENTS
C
    4 CALL BOUND
      GO TO 900
C
C     EXPANSION JOINT ELEMENTS
C
    5 CALL EXPJO
C
  900 CONTINUE
      RETURN
 1001 FORMAT (14I5)
      END


      SUBROUTINE WRITET(MBAND,NDIF)
      COMMON/ELPAR/ NPAR(14),NUMNP,MBANK,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON /EM/LM(24),ND,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
C
C     CALCULATION OF BAND WIDTH AND WRITES ELEMENT MATRICES ON TAPES
C
      MIN=10000C
      MAX=0
      DO 450 L=1,ND
      IF (LM(L).EQ.0) GO TO 450
      IF (LM(L).GT.NEQ) GO TO 450
      IF (LM(L).GT.MAX) MAX=LM(L)
      IF (LM(L).LT.MIN) MIN=LM(L)
  450 CONTINUE
      NDIF=MAX-MIN+1
      IF (NDIF.GT.MBAND) MBAND=NDIF
C
      WRITE (1) NS,ND,LM,ST,TT
      WRITE (2) LM,ND,NS,S,P,XM
C
      RETURN
      END
```

```
      SUBROUTINE BLOCEQ (NEQG,NL,NOYN)
C
C     DETERMINE APPROPRIATE NEQB FOR EACH TYPE OF ANALYSIS
C
      COMMON/ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON/MISC/ NBLOCK,NEQB,LL,NF,LB,NSVV,NFN,NGM,NAT,NT,NOT
      DIMENSION N(3),NB(4)
C
      IF (NOYN.GT.0) GO TO 10
C
C     STATIC ANALYSIS
C
      IF (LL.GE.1) GO TO 5
      WRITE(6,2000)
      STOP
    5 NL=LL
      NEQB=(MTOT-4*LL)/(MBAND+LL+1)/2
      NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL*(2*(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      NB(1)=(MTOT-NSBB)/(LL*MBAND+1)
      IF (NEQB.GT.NB(1)) NEQB=NB(1)
      RETURN
C
C     DYNAMIC ANALYSIS ---- MODES AND FREQUENCIES
C
   10 NL=0
      LL=1
      NEQB=(MTOT-4*LL)/(MBAND+LL+1)/2
      NBLOCK=(NEQ-1)/NEQB+1
      IF (NBLOCK.GT.1) GO TO 12
      N(1)=NEQ*MBAND+(NF+3)*(NEQ+5)+1
      IF (N(1).GT.MTOT) CALL ERROR (N(1)-MTOT)
      GO TO 30
   12 NV=2*NF
      IF (NF.GT.NSVV) NV=NF+NSVV
      NB(1)=MTOT/(2*MBAND+1)
      NB(2)=(MTOT-3*NV*(NV+1))/(2*NV+1)
      NB(3)=(MTOT-6*NV)/(MBAND+1)
      DO 15 I=1,3
   15 IF (NEQB.GT.NB(I)) NEQB=NB(I)
   20 NBLOCK=(NEQ-1)/NEQB+1
      NSB=(MBAND-2)/NEQB+1
      IF (NSB.GE.NBLOCK) NSB=NBLOCK-1
      NB(4)=(MTOT-2*NV-1)/(MBAND+NV+NV*(NSB+1)+1)
      IF (NEQB.LE.NB(4)) GO TO 30
      NEQB=NB(4)
      GO TO 20
   30 IF (NOYN.EQ.1) RETURN
C
C     DYNAMIC ANALYSIS ---- RESPONSE SPECTRUM
C
      IF (NOYN.EQ.2) GO TO 40
      IF(NOYN.EQ.3) LL=NF
      NB(1)=(MTOT-6*NF)/(NF+3)
      NB(2)=(MTOT-6*NUMNP-6*(NF+1))/(NF+2)
      DO 35 I=1,2
      IF (NEQB.GT.NB(I)) NEQB=NB(I)
   35 CONTINUE
      RETURN
C
C     DYNAMIC ANALYSIS ---- TIME HISTORY RESPONSE
C
   40 NOS=(NT-1)/NOT
      N(2)=NFN*(NF+1)+NAT*NT+1
      N(1)=NELTYP+NF*(NOS+8)+1
      MAX=(MTOT-N(2))/2
      IF (NT.GT.MAX) MAX=NT
      N(2)=N(2)+MAX
      N(3)=MTOT
      IF (N(1).GT.N(3)) N(3)=N(1)
      IF (N(2).GT.N(3)) N(3)=N(2)
      IF (N(3).GT.MTOT) CALL ERROR (N(3)-MTOT)
      NB(1)=(MTOT-NFN*NF*NAT-2*LL )/(2*NFN+NF*(LL+1))
      NB(2)=(MTOT-6*NUMNP)/(NFN+1)
      NB(3)=(MTOT-NELTYP-8*NF)/(1+NBLOCK*NF)
      DO 45 I=1,3
      IF (NEQB.GT.NB(I)) NEQB=NB(I)
   45 CONTINUE
C
      NCN=NGM-1
      IF (NCN) 70, 60, 50
   50 IF (NEQG.GT.NEQ) GO TO 55
      WRITE(6,2001)
      STOP
   55 LL=NEQG-NEQ
      NB(1)=(MTOT-4*LL)/(MBAND+LL+1)/2
      NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL*(2*(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      NB(2)=(MTOT-NSBB)/(LL*MBAND+1)
      NB(3)=(MTOT-2*LL-6*NUMNP)/(LL+2)
      NB(4)=(MTOT-NFN*NF*NAT-2*LL)/(2*NFN+NF*LL+1)
      DO 57 I=1,4
      IF (NEQB.GT.NB(I)) NEQB=NB(I)
   57 CONTINUE
      GO TO 70
C
   60 NB(1)=(MTOT-2*LL)/(NFN+1)/2
      IF (NEQB.GT.NB(1)) NEQB=NB(1)
C
   70 RETURN
C
 2000 FORMAT (////
     . 53H FOR STATIC ANALYSIS LL MAY NOT BE INPUT AS ZERO.....
     . 22H EXECUTION TERMINATED. )
 2001 FORMAT (////
     . 60H JOINT DOF CODES ARE NOT SPECIFIED CORRECTLY FOR JOINTS     /
     . 60H HAVING MULTIPLE GROUND MOTION INPUT...EXECUTION TERMINATED.)
C
      END
```

```
      SUBROUTINE INL(ID,B,TR,TMASS,NUMNP,NEQB,LL,NF,NEQ)
C
C     INPUT NODAL LOADS AND MASSES
C
      DIMENSION ID(NUMNP,6),B(NEQB,LL),TR(6,LL),TMASS(NEQB)
      COMMON / JUNK / R(6),TXM(6)
C
      NT=3
      REWIND NT
      KSHF=0
      IF (NF.NE.0) GO TO 720
      WRITE (6,2002)
      GO TO 740
  720 WRITE (6,2003)
  740 DO 750 I=1,NEQB
      TMASS(I)=0.
      DO 750 K=1,LL
  750 B(I,K)=0.0
C
      DO 900 NN=1,NUMNP
C
      DO 100 I=1,6
      TXM(I)=0.
      DO 100 J=1,LL
  100 TR(I,J)=0.0
C
      IF(NN.EQ.1) GO TO 300
  150 IF(N.NE.NN) GO TO 400
      DO 200 I=1,6
      IF (L) 180,180,190
  180 TXM(I)=R(I)
      GO TO 200
  190 TR(I,L)=R(I)
  200 CONTINUE
  300 READ (5,1001) N,L,R
      IF (N.EQ.0) GO TO 150
      WRITE(6,2001) N,L,R
      GO TO 150
C
  400 DO 800 J=1,6
      II=ID(NN,J)
      IF (II.GT.NEQ) GO TO 800
      II=II-KSHF
      IF (II) 800,800,500
  500 DO 600 K=1,LL
  600 B(II,K)=TR(J,K)
      TMASS(II)=TXM(J)
  610 IF(II.NE.NEQB) GO TO 800
      WRITE (NT) B,TMASS
      KSHF=KSHF+NEQB
      DO 700 I=1,NEQB
      TMASS(I)=0.
      DO 700 K=1,LL
  700 B(I,K)=0.0
  800 CONTINUE
  900 CONTINUE
      WRITE (NT) B,TMASS
C
      RETURN
 1001 FORMAT (2I5,7F10.4)
 2001 FORMAT (2I5,7F12.3)
 2002 FORMAT (23H1.....NODAL POINT LOADS // 10H NODE LOAD 27X
     . 14HAPPLIED LOADS          / 10H  NO. CASE  8X  2HFX  10X
     .   2HFY 10X 2HFZ 10X 2HMX 10X 2HMY 10X 2HMZ )
 2003 FORMAT (23H1.....NODAL POINT MASSES  // 10H NODE LOAD 27X
     . 19HCONCENTRATED MASSES /10H  NO. CASE  8X  2HMX  10X
     .   2HMY 10X 2HMZ 10X 3HI/X 9X 3HI/Y 9X 3HI/Z )
      END
```

```
      SUBROUTINE ADDSTF(A,B,STR,TMASS,NUMEL,NBLOCK,NE2B,LL,MBAND,NL,NEQ)
C
C     FORMS GLOBAL EQUILIBRIUM EQUATIONS IN BLOCKS
C
      DIMENSION A(NE2B,MBAND),B(NE2B,LL),STR(4,LL),TMASS(NE2B),STIF(722)
      COMMON/EM/LM(24),ND,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
      COMMON/MISC/ NBLOC,NEQBL,LLL,NF,LB,NSVV,NFN,NGM,NAT,NT,NOT
      EQUIVALENCE (STIF,LM)
      NEQB=NE2B/2
      K=NEQB+1
      X=NBLOCK
      MB=SQRT(X)
      MB=MB/2+1
      NEBB=MB*NE2B
      MM=1
C
      NSHIFT=0
      REWIND 3
      REWIND 4
      REWIND 9
C
C     READ ELEMENT LOAD MULTIPLIERS
C
      DO 40 L=1,LL
      DO 40 I=1,4
   40 STR(I,L)=0.
C
      IF (NL.EQ.0) GO TO 65
      WRITE (6,2000)
      DO 50 L=1,LL
      READ  (5,1002)    (STR(I,L),I=1,4)
   50 WRITE (6,2002) L,(STR(I,L),I=1,4)
C
   65 CONTINUE
      WRITE (8) STR
C
C     FORM EQUATIONS IN BLOCKS       ( 2 BLOCKS AT A TIME)
C
      DO 1000 M=1,NBLOCK ,2
      DO 100 I=1,NE2B
      DO 100 J=1,MBAND
  100 A(I,J)=0.
      READ (3) ((B(I,L),I=1,NEQB),L=1,LL),(TMASS(I),I=1,NEQB)
      IF (M.EQ.NBLOCK) GO TO 200
      READ (3) ((B(I,L),I=K,NE2B),L=1,LL),(TMASS(I),I=K,NE2B)
  200 CONTINUE
C
      REWIND 7
      REWIND 2
      NA=7
      NUML=NUM7
      IF (MM.NE.1) GO TO 75
      NA=2
      NUME=NUMEL
      NUM7 =0
C
   75 DO 700 N=1,NUME
      READ (NA)  STIF
      DO 600 I=1,ND
      IF (LM(I).GT.NEQ) GO TO 600
      LMN=I-LM(I)
      II=LM(I)-NSHIFT
      IF (II.LE.0.OR.II.GT.NE2B) GO TO 600
      TMASS(II)=TMASS(II)+XM(I)
      IF (NL) 220,220,210
  210 CONTINUE
      DO 300 L=1,LL
      DO 300 J=1,4
  300 B(II,L)=B(II,L)+P(I,J)*STR(J,L)
  220 DO 500 J=1,ND
      JJ=LM(J)
      IF (JJ.LE.NEQ) GO TO 350
      L=JJ-NEQ
      B(II,L)=B(II,L)+S(I,J)
      GO TO 500
  350 JJ=JJ+LMN
      IF(JJ) 500,500,400
  400 A(II,JJ)=A(II,JJ)+S(I,J)
  500 CONTINUE
  600 CONTINUE
C
C     DETERMINE IF STIFFNESS IS TO BE PLACED ON TAPE 7
C
      IF (MM.GT.1) GO TO 700
      DO 650 I=1,ND
      IF (LM(I).GT.NEQ) GO TO 650
      II=LM(I) -NSHIFT
      IF(II.GT.NE2B.AND.II.LE.NEBB) GO TO 660
  650 CONTINUE
      GO TO 700
  660 WRITE (7) STIF
      NUM7=NUM7+1
C
  700 CONTINUE
      WRITE(4) ((A(I,J),I=1,NEQB),J=1,MBAND),((B(I,L),I=1,NEQB),L=1,LL)
      IF (NGM.EQ.2)
     .WRITE(12)((A(I,J),I=1,NEQB),J=1,MBAND),((B(I,L),I=1,NEQB),L=1,LL)
      WRITE(9) (TMASS(I),I=1,NEQB),((B(I,L),I=1,NEQB),L=1,LL)
      IF(M.EQ.NBLOCK) GO TO 1000
      WRITE(4) ((A(I,J),I=K,NE2B),J=1,MBAND),((B(I,L),I=K,NE2B),L=1,LL)
      IF (NGM.EQ.2)
     .WRITE(12)((A(I,J),I=K,NE2B),J=1,MBAND),((B(I,L),I=K,NE2B),L=1,LL)
      WRITE(9) (TMASS(I),I=K,NE2B),((B(I,L),I=K,NE2B),L=1,LL)
      IF (MP.EQ.MP) MM=0
      MM=MM+1
 1000 NSHIFT=NSHIFT+NE2B
C
      RETURN
 1002 FORMAT (4F10.0)
 2000 FORMAT (1CH1STRUCTURE 12X 25HELEMENT LOAD MULTIPLIERS /
     . 10H LOAD CASE 9X 1HA 9X 1HB 9X 1HC 9X 1HD/)
 2002 FORMAT (I16,7X,4F10.3)
      END
```

```
      SUBROUTINE STATIC
C
      COMMON / MISC / NBLOCK,NEQB,LL,NF,LB,NSVV
      COMMON /JUNK/ XXX(4),NDYN,JUK(200)
      DIMENSION T(4)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON A(1)
C
      CALL SECOND (T(1))
C
C     SOLVE FOR DISPLACEMENT UNKNOWNS
C             ( SOLN ON TAPE 2.  TAPES 3 AND 7 USED FOR TEMP STORAGE)
C
      NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL*(2+(MBAND-1)/NEQB)
      IF(NSBB.LT.NSB) NSBB=NSB
      N4=N3+NSBB
      CALL USOL(A(N1),A(N3),A(N4),NEQB,MBAND,LL,NBLOCK,NSB,4,3,7,2,2)
C
C     PRINT DISPLACEMENT
C
      CALL SECOND (T(2))
C
      N2=N1+NUMNP*6
      N3=N2+6*LL
      CALL PRINTD(A(N1),A(N2),A(N3),NEQB,NUMNP,LL,NBLOCK,NEQ,2,0)
C
      CALL SECOND (T(3))
C
C     COMPUTE STRESSES
C
   30 N2=N1+4*LL
      N3=N2+NEQB*LL
      LB=(MTOT-N3)/(NEQ +12)
      NDYN=0
      CALL STRESS(A(N1),A(N2),A(N3),NEQB,LB,LL,NEQ,NBLOCK)
C
      CALL SECOND (T(3))
C
      T(1)=T(2)-T(1)
      T(2)=T(3)-T(2)
      T(3)=T(4)-T(3)
      WRITE (6,1000) T(1),T(2),T(3)
C
      RETURN
 1000 FORMAT (23H1....TIME LOG (SECONDS)  ///
     .         33H  EQUATION SOLVING.............. ,F8.2//
     . 33H  PRINT DISPLACEMENTS...........   , F8.2 //
     . 33H  COMPUTE STRESSES.............    , F8.2 //)
      END
```

```
      SUBROUTINE USOL (A,B,MAXB,NEQB,MB,LL,NBLOCK,NSB,NORG,NBKS,NT1,
     .                                                         NT2,NST)
      DIMENSION A(NSB),B(NSB),MAXB(NEQB)
C
      NC=MB+LL
      NBR=(MB-1)/NEQB+1
      INC=NEQB-1
      NMB=NEQB*MB
      N2=NT2
      N1=NT1
      REWIND NORG
      REWIND NBKS
C
C     REDUCE EQUATIONS BLOCK-BY-BLOCK
C
      DO 900 N=1,NBLOCK
      IF (N.GT.1.AND.NBR.EQ.1) GO TO 110
      IF (NBR.LL.1) GO TO 105
      REWIND N1
      REWIND N2
  105 NI=N1
      IF(N.EQ.1) NI=NORG
      READ (NI) A
  110 DO 300 I=1,NEQB
      D=A(I)
      IF(D) 115,300,120
  115 M=NEQB*(N-1)+I
      WRITE (6,116) M,D
  116 FORMAT (33H0SET OF EQUATIONS MAY BE SINGULAR   /
     .  26H DIAGONAL TERM OF EQUATION   I8, 9H  EQUALS   1PE12.4)
C
  120 II=I
      DO 125 J=2,NC
      II=II+NEQB
  125 A(II)=A(II)/D
C
      DO 130 J=I,NMB,NEQB
      IF (A(J).NE.0.) MAXB(I)=J
  130 CONTINUE
C
      JL=I+1
      IF (JL.GT.NEQB) GO TO 300
      II=I
      DO 200 J=JL,NEQB
      II=II+NEQB
      IF(II.GT.NMB) GO TO 200
      C=A(II)
      IF (C.EQ.0.0) GO TO 200
      C=C*A(I)
C
      KK=J
      MAX=MAXB(I)
      DO 150 JJ=II,MAX,NEQB
      A(KK)=A(KK)-C*A(JJ)
  150 KK=KK+NEQB
C
      KK=J +NMB
      JJ=I+NMB
      DO 175 L=1,LL
      A(KK)=A(KK)-C*B(JJ)
      KK=KK+NEQB
```

```
      WRITE (NBKS) A,MAXB
C
C     SUBSTITUTE INTO REMAINING EQUATIONS
C
      DO 800 NN=1,NBR
      IF(N+NN.GT.NBLOCK) GO TO 800
      NI=N1
      IF(N.EQ.1) NI=NORG
      IF(NN.EQ.NBR) NI=NORG
      READ (NI) B
      IL=1+NN*NEQB*NEQB
      DO 700 I=1,NEQB
      II=IL
      DO 690 K=1,NEQB
      IF (II.GT.NMB) GO TO 690
      C=A(II)
      IF (C.EQ.0.0) GO TO 690
      C=C*A(K)
      MAX=MAXB(K)
C
      KK=I
      DO 640 JJ=II,MAX,NEQB
      B(KK)=B(KK)-C*A(JJ)
  640 KK=KK+NEQB
      KK=I+NMB
      JJ=K+NMB
      DO 650 L=1,LL
      B(KK)=B(KK)-C*A(JJ)
      KK=KK+NEQB
  650 JJ=JJ+NEQB
  690 II=II-INC
  700 IL=IL+NEQB
C
      IF(NBR.NE.1) GO TO 750
      DO 740 I=1,NSB
  740 A(I)=B(I)
      GO TO 800
  750 WRITE (N2) B
  800 CONTINUE
      M=N1
      N1=N2
  900 N2=M
C
C     BACKSUBSTITUTION - RESULTS ON TAPE NRST
C
      LS=LL*NEQB
      NEB=NEQB*(NBR+1)
      NUM=NBR*NEQB
      MAX=NEB*LL
      DO 905 I=1,MAX
  905 B(I)=0.
      REWIND NRST
C
      DO 1000 N=1,NBLOCK
      BACKSPACE NBKS
      READ (NBKS) A,MAXB
      BACKSPACE NBKS
      DO 910 L=1,LL
      K=L*NEB
      DO 910 J=1,NUM
      I=K-NEQB
      B(K)=B(I)
  910 K=K-1
C
      I=NMB
      DO 920 L=1,LL
      K=(L-1)*NEB
      DO 920 J=1,NEQB
      I=I+1
      K=K+1
  920 B(K)=A(I)
C
      DO 955 I=1,NEQB
      J=NEQB+1-I
      MAX=MAXB(J)
      IF (A(J).EQ.0.) GO TO 955
      DO 950 L=1,LL
      KK=J+(L-1)*NEB
      JJ=KK+1
      IL=J+NEQB
      C=B(KK)
      DO 940 II=IL,MAX,NEQB
      C=C-A(II)*B(JJ)
  940 JJ=JJ+1
  950 B(KK)=C
  955 CONTINUE
C
      I=0
      DO 960 L=1,LL
      K=(L-1)*NEB
      DO 960 J=1,NEQB
      K=K+1
      I=I+1
  960 A(I)=B(K)
C
      WRITE (NRST) (A(I),I=1,LS)
 1000 CONTINUE
      RETURN
C
 2000 FORMAT (33H0SET OF EQUATIONS MAY BE SINGULAR   /
     . 26H DIAGONAL TERM OF EQUATION   I4, 8H  EQUALS   1PE12.4)
      END
```

```
      SUBROUTINE PRMAT(A,NR,NC,MAX,TITLE)
C
      DIMENSION A(MAX,1)
C
      PRINT 1002,TITLE
      DO 100 J=1,NC,8
      JH=J+7
      IF (JH-NC) 75,75,50
   50 JH=NC
   75 PRINT 1000,(N,N=J,JH)
      DO 100 I=1,NR
  100 PRINT 1001,I,(A(I,K),K=J,JH)
C
      RETURN
C
 1000 FORMAT (///8X,8I14)
 1001 FORMAT (I4,4X,8E14.7)
 1002 FORMAT (/////6H MATRIX ,A10)
C
      END


      SUBROUTINE PRINTD(ID,D,B,NEQB,NUMNP,LL,NBLOCK,NEQ,NT,NF)
C
      DIMENSION ID(NUMNP,6),B(NEQB,LL),D(6,LL)
C
      REWIND NT
      IF(NF.GT.0) READ (NT)
      REWIND 8
      READ (8) ID
      M=NEQ
      NN=NEQB*NBLOCK
      IF(NF.EQ.0) WRITE (6,2003)
      IF(NF.GT.0) WRITE (6,2005)
      N=NUMNP
C
      DO 500 KK=1,NUMNP
C
      III=6
      JJJ=6
      I=6
      DO 250 II=1,6
      DO 100 L=1,LL
  100 D(I,L)=0.
      IF(M.GT.NN) GO TO 150
      IF (M.EQ.0) GO TO 150
      READ (NT) B
      NN=NN-NEQB
  150 NID=ID(N,I)
      IF (NID.GT.NEQ) GO TO 250
      IF (NID.LT.0) III=III-1
      IF (NID.EQ.0) JJJ=JJJ-1
      IF (NID.LT.1) GO TO 250
      K=M-NN
      M=M-1
C
      DO 200 L=1,LL
  200 D(I,L)=B(K,L)
  250 I=I-1
C
      IF (III.EQ.0) GO TO 400
      IF (JJJ.EQ.0) GO TO 450
      WRITE (6,2004) N,(L,(D(I,L),I=1,6),L=1,LL)
      GO TO 500
C
  400 WRITE (6,2006) N
      GO TO 500
C
  450 WRITE (6,2007) N
C
  500 N=N-1
C
      RETURN
C
 2003 FORMAT (40H1.......NODE DISPLACEMENTS AND ROTATIONS//
     . 5H NODE  5H LOAD  11X  1HX  11X  1HY  11X  1HZ  9X  2HXX
     .  9X  2HYY  9X  2HZZ/5H   NO,5H   NO /)
 2004 FORMAT (1H0,I4,I5,1P3E12.3,3E11.2/(I10,3E12.3,3E11.2))
 2005 FORMAT (19H1.......MODE SHAPES  //
     . 5H0NODE  5H MODE  11X  1HX  11X  1HY  11X  1HZ  9X  2HXX
     .   9X  2HYY  9X  2HZZ/5H   NO,5H   NO /)
 2006 FORMAT (1H0,I4,5X,18H...SLAVE JOINT....  )
 2007 FORMAT (1H0,I4,5X,24H...FIXED JOINT (ALL 0.0) )
C
      END


      SUBROUTINE STRESS (STR,B,D,NEQB,LB,LL,NEQ,NBLOCK)
C
      DIMENSION D(NEQ,LB),B(NEQB,LL),STR(4,LL)
      COMMON/EM/ND,NS,LM(24),S(24,24),P(24,4),XM(24),SA(12,24),TT(12,4)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON / JUNK / MM,L,K,NTAG,NDYN,SIG(12),EXTRA(188)
C
      READ (8) STF
      NT=(LL-1)/LB +1
      LH=0
      REWIND 3
C
      DO 1000 II=1,NT
C
      LT =LH+1
      LLT=1-LT
      LH=LT+LB-1
      IF(LH.GT.LL) LH=LL
C
C     MOVE DISPLACEMENTS INTO CORE FOR LB LOAD CONDITIONS
C
      REWIND 2
      IF(NDYN.EQ.3) READ (2)
      NQ=NEQB*NBLOCK
      DO 200 NN=1,NBLOCK
      READ (2) B
      N=NEQB
      IF (NN.EQ.1) N=NEQ-NQ+NEQB
      NQ=NQ-NEQB
      DO 200 J=1,N
      I=NQ+J
      DO 200 L=LT,LH
```

```
      K=L+LLT
  200 D(I,K)=B(J,L)
      LK=LM-LT+1
C
C     CALCULATE STRESSES FOR ALL ELEMENTS FOR LB LOAD CONDITIONS
C
      REWIND 1
      DO 1000 M=1,NELTYP
C
      IF(NDYN.EQ.0) WRITE (6,2000)
      IF(NDYN.EQ.3) WRITE (6,2001)
C
      READ (1) NPAR
      IF(NDYN.EQ.3) WRITE (3) NPAR
      MTYPE=NPAR(1)
      NPAR(1)=0
C
C     CALCULATE STRESSES
C
  250 NUME=NPAR(2)
      NTAG=0
      DO 1000 MM=1,NUME
      READ (1) NS,NC,LM,SA,TT
      WRITE (6,2003)
      K=0
      DO 950 L=LT,LF
      K=K+1
C
      DO 300 N=1,NS
      SIG(N)=TT(N,1)*STR(1,L)+TT(N,2)*STR(2,L)+TT(N,3)*STR(3,L)+
     .  TT(N,4)*STR(4,L)
      DO 300 J=1,NC
      JJ=LM(J)
      IF (JJ.GT.NEQ) GO TO 300
      IF (JJ) 300,300,290
  290 SIG(N)=SIG(N)+SA(N,J)*D(JJ,K)
  300 CONTINUE
C
C     CALCULATE ADDITIONAL STRESSES AND PRINT RESULTS
C
      GO TO (1,2,3,4,5) MTYPE
C     THREE DIMENSIONAL TRUSS ELEMENTS
C
    1 CALL TRUSS
      GO TO 900
C
C     THREE DIMENSIONAL BEAM ELEMENTS
C
    2 CALL BEAM
      GO TO 900
C
C
C     THREE DIMENSIONAL CURVED BEAM ELEMENTS
C
    3 CALL CBEAM
      GO TO 900
C
C     BOUNDARY ELEMENTS
C
    4 CALL BOUND
      GO TO 900
C
C     EXPANSION JOINT ELEMENTS
C
    5 CALL EXPJB
C
  900 CONTINUE
      IF(NDYN.EQ.3) WRITE (3) NS,SIG
  950 CONTINUE
 1000 CONTINUE
C
      RETURN
 2000 FORMAT (25H1 STATIC STRESS PRINT OUT )
 2001 FORMAT (28H1 RESPONSE SPECTRUM ANALYSIS //
     . 38H  STRESS PRINT OUT IN INDIVIDUAL MODES  )
 2003 FORMAT (/)
      END
```

```
      SUBROUTINE MODES
C
C
      COMMON / MISC / NBLOCK,NEQB,LL,NF,LB,NSVV
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      DIMENSION T(3)
      COMMON A(1)
C
C     EVALUATION OF MODE SHAPES AND FREQUENCIES
C
      CALL SECOND (T(1))
C
      IF (NSVV.LT.10) NSVV=10
      CALL DYNAM (NEQ,MBAND,NBLOCK,NEQB,NF,NSVV,MTOT)
C
C     PRINT MODE SHAPES IN JOINT ORDER
C
C
      CALL SECOND (T(2))
C
      N2=1+6*NUMNP
      N3=N2+6*NF
      CALL PRINTD (A(1),A(N2),A(N3),NEQB,NUMNP,NF,NBLOCK,NEQ,7,NF)
C
      CALL SECOND (T(3))
      T(1)=T(2)-T(1)
      T(2)=T(3)-T(2)
      WRITE (6,3000) T(1),T(2)
C
      RETURN
C
 3000 FORMAT (23H1....TIME LOG (SECOND)  ///
     . 33H  MODE SHAPES AND FREQUENCIES...   ,F8.2//
     . 33H  PRINT MODE SHAPES.............   ,F8.2//)
C
      END


      SUBROUTINE DYNAM (NEQ,MBAND,NBLOCK,NEQB,NF,NSVV,MTOT)
C
C      COMPUTE SPECFIED NUMBER OF SMALLEST EIGENVALUES AND ASSOCIATED
C      VECTORS IN THE GENERALIZED EIGENVALUE PROBLEM....BY K.J. BATHE
C
       COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
       COMMON A(1)
C
      NSTIF=4
      NMASS=9
      NRED=10
      NL=2
      NR=3
      NT=7
      WRITE (6,1000) NEQ,MBAND,NBLOCK,NEQB,NF
C
      IF (NBLOCK.GT.1) GO TO 300
      NIM=2
      NC=NF+NIP
      NWA=NEQ*MBAND
      N2=1+NWA
      N3=N2+NEQ
      N4=N3+NEQ
      N5=N4+NEQ
      N6=N5+NEQ
      N7=N6+NEQ*NC
      N8=N7+NEQ*NC
      N9=N8+NC
      N10=N9+NC
      N11=N10+NC
      N12=N11+NC
      N13=N12+NC
C
      IF (MTOT-N13) 100,200,200
  100 PH=7HSECANTC
      WRITE (6,1010) PH,N13
      STOP
C
  200 CALL SECANTC (A(1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9
     1),A(N10),A(N11),A(N12),NEQ,MBAND,NWA,NF,NC)
      DO 250 I=1,NF
      II=N8+I-1
  250 A(I)=A(II)
      GO TO 600
C
  300 NWA=NEQB*MBAND
      NV=2*NF
      IF (NF.GT.NSVV) NV=NF+NSVV
      NWV=NV*NEQB
      NTB=(MBAND-2)/NEQB+1
      IF (NTB.GE.NBLOCK) NTB=NBLOCK 1
      NWVV=NWV*(NTB+1)
      N=2*NWA+NEQB
      N2=NWV+NEQB+NV
      N3=NWA+NWV+NWVV+NEQB+2*NV
      N4=NV*(3*NV+2*NEQB+3)+NEQB
      N5=NWA+6*NV+NEQB
C
      IF (N2.GT.N) N=N2
      IF (N3.GT.N) N=N3
      IF (N4.GT.N) N=N4
      IF (N5.GT.N) N=N5
      IF (MTOT N) 400,500,500
  400 PH=7HSSPACEB
      WRITE (6,1010) PH,N
      STOP
C
  500 CALL SSPACEB (NEQ,MBAND,NBLOCK,NEQB,NF,NV,NWA,NWV,NWVV,NTB)
C
  600 RETURN
C
 1000 FORMAT (1H1,30HFREQUENCY ANALYSIS INFORMATION  //
     1          /30H NO OF EQUATIONS .............   I4
     2          /30H 1/2 BANDWIDTH OF A ..........   I4
     3          /30H NO OF BLOCKS ................   I4
     4          /30H NO OF EQNS PER BLOCK ........   I4
     5          /30H NO OF FREQUENCIES REQD ......   I4  )
 1010 FORMAT (30HOFOR EXECUTION OF SUBROUTINE  ,A7,30H MTOT NEED TO BE I
     .NCREASE TO  ,I6)
C
      END
```

```
      SUBROUTINE SECANTD (A,B,V,MAXA,W,VV,WW,ROOT,TIM,ERRVL,ERRVR,
     *                    NITE,N,MA,NWA,NROOT,NC)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      DIMENSION A(NWA),B(N),V(N),VS(1),W(N),VV(N,NC),WW(N,NC),ROOT(NC),
     *          TIM(NC),ERRVL(NC),ERRVR(NC)
      INTEGER NITE(NC),MAXA(NC)
C
C     FOLLOWING TOLERANCES ARE SET FOR CDC 6400.....
C
      ACTOL=1.0E-04
      RCBTOL=1.0E-06
      RTOL=1.0E-10
      RQTOL=1.0E-12
      SCALE=2.0**SCO
C
      NTF=5
      IITEM=10
      NITEM=40
C
      REWIND NMASS
      READ (NMASS) B
C
      ETA=2.0
      NOV=0
      JR=1
      NSK=0
      NWA=N*MA
      ISC=1000
C
      CALL SECOND (TIM1)
      RA=0.0
      RB=0.0
      CALL BANDET (A,B,V,MAXA,N,NWA,RA,NSCH,DETA,ISC,1)
      FA=DETA
      FB=FA
      DETB=DETA
C
C     FIND LOWER BOUND ON SMALLEST EIGENVALUE
C
      WRITE (6,1010)
      DO 100 I=1,N
  100 W(I)=B(I)
      RT=0.0
      IITE=0
      KK=3
  110 IITE=IITE+1
      DO 120 I=1,N
  120 V(I)=W(I)
      CALL BANDET (A,B,V,MAXA,N,NWA,RA,NSCH,DETA,ISC,KK)
      KK=2
      RQT=0.0
      DO 130 I=1,N
  130 RQT=RQT+W(I)*V(I)
      DO 180 I=1,N
  180 W(I)=B(I)*V(I)
      RQB=0.0
      DO 140 I=1,N
  140 RQB=RQB+W(I)*V(I)
      RQ=RQT/RQB
      WRITE (6,1004) RQ
      BS=SQRT(RQB)
      TOL=ABS(RQ-RT)/RQ
      IF (TOL.LT.RQTOL) GO TO 150
      DO 160 I=1,N
  160 W(I)=W(I)/BS
      RT=RQ
      IF (IITE.LT.IITEM) GO TO 110
C
  150 DO 170 I=1,N
  170 V(I)=V(I)/BS
      RB=RQ*(1.0-AMIN1(0.1,100*TOL))
      IS=0
  230 CALL BANDET (A,B,V,MAXA,N,NWA,RB,NSCH,DETB,ISC,1)
      WRITE (6,1020) RB,NSCH
      FB=DETB
      IF (NSCH.EQ.0) GO TO 300
      IS=IS+1
      IF (IS.LE.NTF) GO TO 240
      WRITE (6,1030)
      STOP
  240 RB=RB/(NSCH+1)
      GO TO 230
C
C     ITERATION FOR INDIVIDUAL ROOT
C
  300 WRITE (6,1040)
      NITE(JR)=1
      WRITE (6,1050) JR,NITE(JR),RA,DETA,FA,ETA,ISC
      NITE(JR)=2
      WRITE (6,1050) JR,NITE(JR),RB,DETB,FB,ETA,ISC
C
C     STOP WHEN REQUIRED NO. OF ROOTS SMALLER THAN RC AND NOV=0 FOUND
C
  310 IF (NSCH.GE.NROOT) GO TO 900
C
      DIF=FB-FA
      IF (DIF.NE.0.0) GO TO 320
      WRITE (6,1060)
      GO TO 900
  320 DEL=FB*(RB-RA)/DIF
      RC=RB-ETA*DEL
      TOL=RCBTOL*RC
      IF (ABS(RC-RB).GT.TOL) GO TO 330
      WRITE (6,1070)
      ROOT(JR)=RB
      GO TO 400
C
  330 CALL BANDET (A,B,V,MAXA,N,NWA,RC,NSCH,DETC,ISC,1)
      FC=DETC
      NITE(JR)=NITE(JR)+1
      IF (JR.EQ.1) GO TO 340
      JJ=JR-1
      DO 350 K=1,JJ
  350 FC=FC/(RC-ROOT(K))
  340 WRITE (6,1050) JR,NITE(JR),RC,DETC,FC,ETA,ISC
C
C     START INVERSE ITERATIONS
C
      NES=0
      IF (JR.EQ.1) GO TO 380
      DO 360 I=1,JJ
  360 IF (ROOT(I).LT.RC) NES=NES+1
```

```
  380 NOV=NSCH-NES
      IF (NOV.EQ.0) GO TO 370
      WRITE (6,1080) NOV
      ROOT(JR)=RC
      IF (NOV.GT.1) NSK=1
C
      GO TO 400
  370 RR=RA
      FR=FA
      DETR=DETA
      RA=RB
      FA=FB
      DETA=DETB
      RB=RC
      FB=FC
      DETB=DETC
C
C     RESET ETA IF NECESSARY
C
      TOL=RB*ACTOL
      IF (ABS(RA-RB).LT.TOL) ETA=ETA*2
      IF (NITE(JR).LE.NITEM) GO TO 310
      WRITE (6,1015) NITE(JR),JR
      GO TO 900
C
C     CHECK FOR STORAGE
C
  400 IF (JR.LE.NC) GO TO 405
      WRITE (6,1050)
      GO TO 900
C
  405 NOR=JR-1
      CALL SECOND (TIM3)
      WRITE (6,1100) NOR
      IF (JR.EQ.1) GO TO 410
      DO 420 I=1,N
  420 V(I)=1.0
      KK=3
  410 DO 430 I=1,N
  430 W(I)=B(I)*V(I)
      IS=0
      RTA=C.0
      GO TO 510
C
C     INVERSE ITERATION
C
  440 NITE(JR)=NITE(JR)+1
      DO 450 I=1,N
  450 V(I)=W(I)
      CALL BANDET (A,B,V,MAXA,N,NWA,RC,NSCH,DETC,ISC,KK)
      IF (IS.EQ.1) GO TO 460
      KK=2
      RQT=C.0
      DO 470 I=1,N
  470 RQT=RQT+W(I)*V(I)
      DO 475 I=1,N
  475 W(I)=B(I)*V(I)
      RQB=C.0
      DO 480 I=1,N
  480 RQB=RQB+W(I)*V(I)
      RQ=RQT/RQB
      RT=RCCT(JR)+RQ
      WRITE (6,1110) JR,NITE(JR),RT,RQ
      TOL=RT*RTOL
      IF (ABS(RT-RTA).GT.TOL) GO TO 510
      IS=1
      GO TO 440
C
  510 RTA=RT
      BS=SQRT(RQB)
      DO 490 I=1,N
  490 W(I)=W(I)/BS
      IF (NOR.EQ.C) GO TO 550
      DO 520 K=1,NOR
      AL=C.C
      DO 530 I=1,N
  530 AL=AL+VV(I,K)*W(I)
      DO 540 I=1,N
  540 W(I)=W(I)-AL*WW(I,K)
  520 CONTINUE
C
  550 IF (NITE(JR).LE.NITEM) GO TO 440
      WRITE (6,1015) NITE(JR),JR
      GO TO 900
C
  460 RQT=C.0
      ERRT=RQB
      DO 570 I=1,N
  570 RQT=RQT+V(I)*W(I)
      DO 560 I=1,N
  560 W(I)=B(I)*V(I)
      RQB=0.0
      DO 580 I=1,N
  580 RQB=RQB+V(I)*W(I)
C
C     OBTAIN A RATHER LARGE ERROR BOUND
C
      RQ=RQT/RQB
      ROOT(JR)=RCCT(JR)+RQ
      ERR=SQRT(ERRT/RQB)
      ERRVL(JR)=RCCT(JR)-ERR
      ERRVR(JR)=RCCT(JR)+ERR
C
      BS=SQRT(RQB)
      DO 590 I=1,N
      W(I)=W(I)/BS
  590 V(I)=V(I)/BS
      DO 600 I=1,N
      WW(I,JR)=W(I)
  600 VV(I,JR)=V(I)
C
      CALL SECOND (TIM2)
      TIM3=TIM2-TIM3
      WRITE (6,1120) TIM3
      TIM(JR)=TIM2-TIM1
      TIM1=TIM2
C
C     DECIDE STRATEGY FOR ITERATION TOWARD NEXT ROOT
C
      TOL=RTOL*ROOT(JR)
      IF (NOV.GT.C) GO TO 700
      IF (ABS(RCCT(JR)-RB).GT.TOL) GO TO 710
      IF (RA.GT.0.C) GO TO 720
      RA=RB/2.
```

```
      CALL BANDET (A,B,V,MAXA,N,NWA,RA,NSCH,DETA,ISC,1)
      FA=DETA
20    RB=RA
      FB=FA
      DETB=DETA
      RA=RR
      FA=FR
      DETA=DETR
      GO TO 710

00    IF (ROOT(JR).GT.RC) NSK=1
      IF (NSK.EQ.1) GO TO 730
      IF (ABS(RC-ROOT(JR)).LT.TCL) GO TO 740
      IF (ABS(ROOT(JR)-RB).LT.TCL) GO TO 750
      RA=RB
      FA=FB
      DETA=DETB
50    RB=RC
      FB=FC
      DETB=DETC
      GO TO 710
40    IF (ABS(ROOT(JR)-RB).GT.TCL) GO TO 710
      IF (RA.GT.0.0) GO TO 760
      RA=RB/2.
      CALL BANDET (A,B,V,MAXA,N,NWA,RA,NSCH,DETA,ISC,1)
      FA=DETA
50    RB=RA
      FB=FA
      DETB=DETA
      RA=RB
      FA=FB
      DETA=DETB
10    FA=FA/(RA-ROOT(JR))
      FB=FB/(RB-ROOT(JR))
      JR=JR+1
      ETA=2.0
      GO TO 300

30    IF (RA.GT.0.0) GO TO 780
      RA=RB/2.
      CALL BANDET (A,B,V,MAXA,N,NWA,RA,NSCH,DETA,ISC,1)
      FA=DETA
80    IF (ABS(ROOT(JR)-RB).GT.TCL) GO TO 770
      RB=RA
      FB=FA
      DETB=DETA
      RA=RR
      FA=FR
      DETA=DETR
70    FA=FA/(RA-ROOT(JR))
      FB=FB/(RB-ROOT(JR))
      FR=FR/(RR-ROOT(JR))
      IF (ROOT(JR).LE.RC) NOV=NOV-1
      JR=JR+1
      NITE(JR)=0
      ROOT(JR)=RC
      IF (NOV.GT.0) GO TO 400
      NSK=0
      ETA=2.0
      GO TO 300

00    NROOT=JR-1
```

```
      IF (NROOT.EQ.0) RETURN
      WRITE (6,1130)
      WRITE (6,1004) (ROOT(J),J=1,NROOT)
      WRITE (6,1140)
      WRITE (6,1006) (NITE(J),J=1,NROOT)
      WRITE (6,1150)
      WRITE (6,1008) (TIM(J),J=1,NROOT)
      WRITE (6,1160)
      WRITE (6,1004) (ERRVL(J),J=1,NROOT)
      WRITE (6,1004) (ERRVR(J),J=1,NROOT)
C
C     ARRANGE EIGENVALUES AND VECTORS IN ASCENDING ORDER
C
      IF (JR.EQ.2) GO TO 950
      JR=JR-2
910   IS=0
      DO 920 I=1,JR
      IF (ROOT(I+1).GE.ROOT(I)) GO TO 920
      IS=IS+1
      RT=ROOT(I+1)
      ROOT(I+1)=ROOT(I)
      ROOT(I)=RT
      DO 930 K=1,N
      RT=VV(K,I+1)
      VV(K,I+1)=VV(K,I)
930   VV(K,I)=RT
920   CONTINUE
      IF (IS.GT.0) GO TO 910
C
950   WRITE (6,1170)
      NROOT=NSCH
      DO 960 I=1,NROOT
      IF (ROOT(I).LE.0.0) GO TO 960
      ROOT(I)=SQRT(ROOT(I))
960   CONTINUE
      WRITE (6,3000) NT
      IF (NT.NE.7) NT=7
      REWIND NT
      WRITE (NT) (ROOT(I),I=1,NROOT)
C
C     PRINT FREQUENCIES AND MODE SHAPES
C
      WRITE (6,2000)
      DO 990 I=1,NROOT
      PERIOD=6.2831853/ROOT(I)
      WRITE (6,2001) I,ROOT(I),PERIOD
990   ROOT(I)=PERIOD
C
      WRITE (NT) ((VV(I,J),I=1,N),J=1,NROOT)
      RETURN
C
1002  FORMAT (1H ,12E11.4)
1004  FORMAT (1H0,6E20.12)
1006  FORMAT (1H0,6I20)
1008  FORMAT (1H0,6F20.2)
1010  FORMAT (1H1,63HINVERSE ITERN GIVES FOLLOWING APPROXIMATN TO LOWES
     1T EIGENVALUE  )
1015  FORMAT (41HCWE ABANDON ITERN BECAUSE NO OF ITERN IS I3,9H FOR ROW
     1T I3 )
1020  FORMAT (5HORB = E20.12,7H NSCH = I4)
1030  FORMAT (30HOWE BETTER CHECK THE MATRICES  )
1040  FORMAT (1H1,4X,4HROOT,4X,4HNITE,18X,2HRC,15X,12HDET (A-RC*B),15X,
```

```
SECA 372           /2HFC,13X,3HETA,4X,3HISC)
SECA 373   1050  FORMAT (1HO,4X,I4,4X,I4,8X,3E22.14,F7.2,I6)
SECA 374   1060  FORMAT (42HOTHE DEFLATED POLYNOMIAL HAS NO MORE ROOTS  )
SECA 375   1070  FORMAT (29HO(RC-RB) IS SMALLER THAN TOL )
SECA 376   1080  FORMAT (16HOWE JUMPED OVER I4,16H UNKNOWN ROOT(S)  )
SECA 377   1090  FORMAT (1H1,36HNO MORE STORAGE FOR VECTORS  WE QUIT  )
SECA 378   1100  FORMAT (1HO,34X,4HROOT,18X,2HRC,18X,4HNCR=,I2)
SECA 379   1110  FORMAT (1HO,4X,I4,4X,I4,8X,2F22.14)
SECA 380   1120  FORMAT (20HOTIME FOR INV ITERN F5.2)
SECA 381   1130  FORMAT (20HOTHE EIGENVALUES ARE  /)
SECA 382   1140  FORMAT (42HONO OF ITERATIONS FOR EACH EIGENVALUE ARE   /)
SECA 383   1150  FORMAT (30HOTIME USED FOR EACH EIGENVALUE /)
SECA 384   1160  FORMAT (43HOFOLLOWING ARE ERROR BOUNDS ON EIGENVALUES )
SECA 385   1170  FORMAT (1H1,62HWE ACCEPT FOLLOWING FREQUENCIES AND MODES ARRANGED
SECA 386        . IN ORDER      )
SECA 387   2000  FORMAT (39H1......PRINT OF FREQUENCIES AND PERIODS//
SECA 388        .  6X,4HMODE,5X,11HFREQUENCIES,13X,7HPERIODS /
SECA 389        .  6X,4H NO ,5X,11H (RAD/SEC) ,13X,7H (SEC)  /)
SECA 390   2001  FORMAT (I10,2F20.4)
SECA 391   3000  FORMAT (////46H EIGENVALUE AND EIGENVECTOR ARE STORED IN TAPE,I3)
SECA 392  C
SECA 393         END


SECA 394         SUBROUTINE BANDET (A,B,V,MAXA,NN,NWA,RA,NSCH,DET,ISCALE,KK)
SECA 395  C
SECA 396         COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
SECA 397         DIMENSION A(NWA),B(1),V(1),MAXA(1)
SECA 398  C
SECA 399  C      TRIANGULARIZE BANDED STIFFNESS MATRIX
SECA 400  C
SECA 401         NR=NN-1
SECA 402         IF (KK-2) 100,700,800
SECA 403  C
SECA 404    100  TOL=1.0E+07
SECA 405         RTOL=1.0E-10
SECA 406         SCALE=2.0**900
SECA 407         NTF=3
SECA 408         IS=1
SECA 409    120  REWIND NSTIF
SECA 410         READ (NSTIF) A
SECA 411         DO 140 I=1,NN
SECA 412    140  A(I)=A(I)-RA*B(I)
SECA 413    160  IF (NWA.EQ.NN) GO TO 230
SECA 414         DO 200 N=1,NR
SECA 415         IH=N+NWA-NN
SECA 416    210  IF (A(IH)) 220,215,220
SECA 417    215  IH=IH-NN
SECA 418         GO TO 210
SECA 419    220  MAXA(N)=IH
SECA 420         PIV=A(N)
SECA 421         IF (PIV) 221,226,221
SECA 422    221  IL=N+NN
SECA 423         L=N
SECA 424         DO 240 I=IL,IH,NN
SECA 425         L=L+1
SECA 426         C=A(I)
SECA 427         IF (C) 225,240,225
SECA 428    225  C=C/PIV
SECA 429         IF (ABS(C).LT.TOL) GO TO 235
SECA 430    226  IS=IS+1
SECA 431         IF (IS.LE.NTF) GO TO 245
SECA 432         WRITE (6,1000) NTF
SECA 433         STOP
SECA 434    245  RA=RA*(1.0-RTOL)
SECA 435         GO TO 120
SECA 436    235  J=L-I
SECA 437         DO 260 K=I,IH,NN
SECA 438    260  A(K+J)=A(K+J)-C*A(K)
SECA 439         A(I)=C
SECA 440    240  CONTINUE
SECA 441    200  CONTINUE
SECA 442    230  IF (A(NN).NE.0.0) GO TO 280
SECA 443         AA=ABS(A(1))
SECA 444         DO 290 I=2,NR
SECA 445    290  AA=AA+ABS(A(I))
SECA 446         A(NN)=-(AA/NR)*1.0E-16
SECA 447  C
SECA 448    280  NSCH=0
SECA 449         ISC=0
SECA 450         DET=1.0
SECA 451         DO 300 I=1,NN
SECA 452         IF (ABS(DET).LT.SCALE) GO TO 320
SECA 453         DET=DET/SCALE
SECA 454         ISC=ISC+1
SECA 455    320  DET=DET*A(I)
SECA 456    300  IF (A(I).LT.0.) NSCH=NSCH+1
SECA 457  C
SECA 458         IF (ISCALE.LT.1000) GO TO 340
SECA 459         ISCALE=ISC
SECA 460         GO TO 900
SECA 461    340  IF (ISC-ISCALE) 350,900,370
SECA 462    350  DET=DET/SCALE
SECA 463         GO TO 900
SECA 464    370  DET=DET*SCALE
SECA 465         GO TO 900
SECA 466  C
SECA 467    700  IL=NN
SECA 468         DO 400 N=1,NR
SECA 469         C=V(N)
SECA 470         V(N)=C/A(N)
SECA 471         IF (NWA-NN) 410,400,410
SECA 472    410  IL=IL+1
SECA 473         IH=MAXA(N)
SECA 474         K=N
SECA 475         DO 420 I=IL,IH,NN
SECA 476         K=K+1
SECA 477    420  V(K)=V(K)-C*A(I)
SECA 478    400  CONTINUE
SECA 479         V(NN)=V(NN)/A(NN)
SECA 480  C
SECA 481    800  IF (NWA-NN) 430,900,430
SECA 482    430  N=NN
SECA 483         DO 440 L=2,NN
SECA 484         N=N-1
SECA 485         IL=N+NN
SECA 486         IH=MAXA(N)
SECA 487         K=N
SECA 488         DO 460 I=IL,IH,NN
SECA 489         K=K+1
SECA 490    460  V(N)=V(N)-A(I)*V(K)
SECA 491    440  CONTINUE
```

```
  900 RETURN
 1000 FORMAT (1H1,20H TRIANG FACTORIZATN I3,32H TIMES ABONDONED,CHECK
     1MATRICES  )
      END
```

```
      SUBROUTINE SSPACEB (NEQ,MBAND,NBLOCK,NEQB,NF,NV,NWA,NWV,NWVV,NTB)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      COMMON A(1)
      LOGICAL CHECK
C
      NITEM=12
      CHECK=.TRUE.
C
C     FACTORIZE STIFFNESS MATRIX
C
      N2=1+NWA
      N3=N2+NWA
      CALL SECOND (TIM1)
      CALL DECOMP (A(1),A(N2),A(N3),NEQB,MBAND,NBLOCK,NWA,NTB,NSCH,NEW)
      CALL SECOND (TIM2)
C
C     ESTABLISH STARTING TRANSFORMATION VECTORS ON TAPE NR
C
      N2=1+NWV
      N3=N2+NEQB
      CALL INVECT (A(1),A(N2),A(N3),NBLOCK,NEQB,NV)
      CALL SECOND (TIM3)
      TIM1=TIM2-TIM1
      TIM2=TIM3-TIM2
      WRITE (6,1000) TIM1
      WRITE (6,1010) TIM2
C
C     PERFORM SUBSPACE ITERATION
C
      DO 100 I=1,NV
  100 A(I)=0.0
      NITE=0
  200 NITE=NITE+1
      WRITE (6,1020) NITE
      CALL SECOND (TIM1)
      N1=1+2*NV
      N2=N1+NWA
      N3=N2+NWV
      N4=N3+NWVV
      CALL REDBAK (A(N1),A(N2),A(N3),A(N4),NEQB,NV,NWA,NWV,NWVV,NTB,
     . NBLOCK)
C
C     SOLVE SUBSPACE EIGENVALUE PROBLEM
C
      N2=1+NV
      N3=N2+NV
      N4=N3+NV*NV
      N5=N4+NV*NV
      N6=N5+NV*NV
      N7=N6+NWV
      N8=N7+NWV
      N9=N8+NV
      CALL SECOND (TIM2)
      CALL EIGSOL (A(1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9)
     . ,NF,NV,NBLOCK,NEQB,NITE)
      CALL SECOND (TIM3)
      TIM1=TIM2-TIM1
      TIM2=TIM3-TIM2
      WRITE (6,1030) TIM1
      WRITE (6,1040) TIM2
```

```
C
      IF (NITE.LT.NITEM) GO TO 200
C
C     PRINT FREQUENCIES
C
      WRITE (6,1050)
      WRITE (6,1060) (A(I),I=1,NF)
      WRITE (6,1070)
      NF1=NV+1
      NF2=NV+NF
      WRITE (6,1060) (A(I),I=NF1,NF2)
C
C     FREQUENCIES AND MODE SHAPES STORED ON TAPE NR
C
      WRITE (6,2020) NR
C
      IF (.NOT.CHECK) GO TO 600
C
C     APPLY STURM SEQUENCE CHECK
C
      CALL SECOND (TIM1)
      N2=1+NV
      N3=N2+NV
      N4=N3+NWA
      N5=N4+NEQB
      N6=N5+NV
      N7=N6+NV
      N8=N7+NV
      CALL SCHECK (A(1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),NWA,
     . NEQB,NBLOCK,NF,NV,SHIFT,NEI)
      WRITE (6,1085) SHIFT
      N2=1+NWA
      N3=N2+NWA
      CALL DECOMP (A(1),A(N2),A(N3),NEQB,MBAND,NBLOCK,NWA,NTB,NSCH,NEQ)
      IF (NSCH.EQ.NEI) GO TO 500
      NSCH=NSCH-NEI
      WRITE (6,1090) NSCH
      GO TO 540
  500 WRITE (6,1100) NSCH
  540 CALL SECOND (TIM2)
      TIM2=TIM2-TIM1
      WRITE (6,1110) TIM2
C
      WRITE (6,2010)
      WRITE (6,2011)
C
      REWIND NR
      READ (NR) (A(I),I=1,NF)
      DO 545 N=1,NBLOCK
  545 READ (NR)
C
      DO 550 I=1,NF
      II=NF+I-1
      A(II)=A(I)
      A(I)=6.283185297/A(I)
  550 WRITE (6,2012) I,A(II),A(I)
C
  600 RETURN
C
 1000 FORMAT (////46H0TIME FOR STIFFNESS FACTORIZATION..............
     .        F6.2)
 1010 FORMAT (46H0TIME FOR GENERATION OF INITIAL TR-VECTORS....  F6.2)
 1020 FORMAT (1H1,11HNO OF ITERN  I4//)
 1030 FORMAT (30H0TIME USED IN ITERN STEP......  F6.2)
 1040 FORMAT (30H0TIME FOR EIGENVALUE SOLN.....  F6.2)
 1050 FORMAT (1H1,26HTHE FINAL EIGENVALUES ARE /)
 1060 FORMAT (1H0,6E22.14)
 1070 FORMAT (//1H ,30HREL. TOL. REACHED BY EIGENVALUES ARE  /)
 1085 FORMAT (///1H0,37HSTURM SEQUENCE CHECK APPLIED AT SHIFT  E22.14)
 1090 FORMAT (10H0THERE ARE I4,21H EIGENVALUES MISSING  )
 1100 FORMAT (20H0WE FOUND THE LOWEST I4,12H EIGENVALUES  )
 1110 FORMAT (36H0TIME FOR STURM SEQUENCE CHECK......  F6.2)
 2010 FORMAT (39H1......PRINT OF FREQUENCIES AND PERIODS  )
 2011 FORMAT (//6X,4HMODE,9X,11HFREQUENCIES,13X,7HPERIODS /
     .        6X,4H NO ,9X,11H (RAD/SEC) ,13X,7H (SEC)  /)
 2012 FORMAT (I10,2F20.8)
 2020 FORMAT (////61H .....FREQUENCIES AND MODE SHAPES IN BLOCK ARE ST
     .RED IN TAPE  ,I3////)
      END



      SUBROUTINE DECOMP (A,B,MAXB,NEQB,MBAND,NBLOCK,NWA,NTB,NSCH,NEQ)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      DIMENSION A(NWA),B(NWA),MAXB(NEQB)
C
C     FACTORIZE STIFFNESS MATRIX
C
      NEQB1=NEQB-1
      N1=NL
      N2=NR
      REWIND NSTIF
      REWIND NRED
      REWIND N1
      REWIND N2
      NSCH=0
C
      DO 600 N=1,NBLOCK
      IF (N.NE.1) GO TO 10
      READ (NSTIF) A
      GO TO 110
   10 IF (NTB.EQ.1) GO TO 110
      REWIND N1
      REWIND N2
      READ (N1) A
C
C     FACTORIZE LEADING BLOCK
C
  110 DO 300 I=1,NEQB1
      PIV=A(I)
      IF (PIV) 120,115,130
  115 II=(N-1)*NEQB+I
      IF (II.GT.NEQ) GO TO 520
      WRITE (6,1000) II
      STOP
  120 NSCH=NSCH+1
  130 IH=I+NWA-NEQB
  140 IF (A(IH)) 160,150,160
  150 IH=IH-NEQB
      GO TO 140
  160 MAXB(I)=IH
```

```
SSPA 182          JL=I+1
SSPA 183          II=I
SSPA 184          DO 200 J=JL,NEQB
SSPA 185          II=II+NEQB
SSPA 186          IF (II-NWA) 170,170,300
SSPA 187    170   C=A(II)
SSPA 188          IF (C) 180,200,180
SSPA 189    180   C=C/PIV
SSPA 190          KK=J
SSPA 191          MAX=MAXB(I)
SSPA 192          DO 250 JJ=II,MAX,NEQB
SSPA 193          A(KK)=A(KK)-C*A(JJ)
SSPA 194    250   KK=KK+NEQB
SSPA 195    200   A(II)=C
SSPA 196    300   CONTINUE
SSPA 197          IF (A(NEQB)) 80,60,70
SSPA 198    60    II=N*NEQB
SSPA 199          IF (II.GT.NEQ) GO TO 520
SSPA 200          WRITE (6,1000)II
SSPA 201          STOP
SSPA 202    80    NSCH=NSCH+1
SSPA 203    70    DO 50 J=NEQB,NWA,NEQB
SSPA 204    50    IF (A(J).NE.0.0) MAXB(NEQB)=J
SSPA 205    C
SSPA 206    C     CARRY OVER INTO TRAILING BLOCKS
SSPA 207    C
SSPA 208          DO 400 NN=1,NTB
SSPA 209          IF ((NN+N).GT.NBLOCK) GO TO 400
SSPA 210          NI=NI
SSPA 211          IF ((N.EQ.1).OR.(NN.EQ.NTB)) NI=NSTIF
SSPA 212          READ (NI) B
SSPA 213          IL=1+NN*NEQB*NEQB
SSPA 214          DO 420 I=1,NEQB
SSPA 215          II=IL
SSPA 216          DO 440 K=1,NEQB
SSPA 217          IF (II-NWA) 410,410,440
SSPA 218    410   C=A(II)
SSPA 219          IF (C) 430,440,430
SSPA 220    430   C=C/A(K)
SSPA 221          MAX=MAXB(K)
SSPA 222          KK=I
SSPA 223          DO 460 JJ=II,MAX,NEQB
SSPA 224          B(KK)=B(KK)-C*A(JJ)
SSPA 225    460   KK=KK+NEQB
SSPA 226          A(II)=C
SSPA 227    440   II=II-NEQB1
SSPA 228    420   IL=IL+NEQB
SSPA 229          IF (NTB.NE.1) GO TO 480
SSPA 230          WRITE (NRED) A,MAXB
SSPA 231          DO 500 I=1,NWA
SSPA 232    500   A(I)=B(I)
SSPA 233          GO TO 600
SSPA 234    480   WRITE (N2) B
SSPA 235    400   CONTINUE
SSPA 236          M=N1
SSPA 237          N1=N2
SSPA 238          N2=M
SSPA 239    520   WRITE (NRED) A,MAXB
SSPA 240    600   CONTINUE
SSPA 241    C
SSPA 242          RETURN
SSPA 243    1000  FORMAT (22HOPIVOT IS ZERO IN ROW  I4)
SSPA 244          END



SSPA 245          SUBROUTINE INVECT (VA,XM,IEQ,NBLOCK,NEQB,NV)
SSPA 246    C
SSPA 247          COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
SSPA 248          DIMENSION VA(NEQB,NV),XM(NEQB),IEQ(1)
SSPA 249    C
SSPA 250          NV1=NV-1
SSPA 251          KK=1
SSPA 252          IND=0
SSPA 253    90    NBV=KK*((NV1-1)/NBLOCK+1)
SSPA 254          IF (NBV.GT.NEQB) NBV=NEQB
SSPA 255          IF (NBV.EQ.NEQB) IND=1
SSPA 256          NBVN=0
SSPA 257          ICOUNT=0
SSPA 258          LL=0
SSPA 259    C
SSPA 260          REWIND NMASS
SSPA 261          REWIND NSTIF
SSPA 262    60    READ (NMASS) XM
SSPA 263          READ (NSTIF) (VA(I),I=1,NEQB)
SSPA 264          ICOUNT=ICOUNT+1
SSPA 265          DO 20 I=1,NEQB
SSPA 266          IF (VA(I).EQ.0.0) GO TO 20
SSPA 267          VA(I)=XM(I)/VA(I)
SSPA 268    20    CONTINUE
SSPA 269    C
SSPA 270          NNV=NEQB/NBV
SSPA 271          DO 40 L=1,NBV
SSPA 272          RT=0.0
SSPA 273          NN=L*NNV
SSPA 274          DO 34 I=L,NN
SSPA 275          IF (VA(I).LT.RT) GO TO 34
SSPA 276          RT=VA(I)
SSPA 277          IJ=I
SSPA 278    34    CONTINUE
SSPA 279          DO 30 I=NN,NEQB
SSPA 280          IF (VA(I).LE.RT) GO TO 30
SSPA 281          RT=VA(I)
SSPA 282          IJ=I
SSPA 283    30    CONTINUE
SSPA 284          IF (VA(IJ).NE.0.0) GO TO 32
SSPA 285          NBVN=NBVN+1
SSPA 286          GO TO 40
SSPA 287    32    LL=LL+1
SSPA 288          IEQ(LL)=(ICOUNT-1)*NEQB+IJ
SSPA 289          IF (LL.GE.NV1) GO TO 50
SSPA 290          VA(IJ)=0.0
SSPA 291    40    CONTINUE
SSPA 292          IF (IND.EQ.1) GO TO 45
SSPA 293          IF ((NBVN.EQ.0).OR.(ICOUNT.EQ.NBLOCK)) GO TO 45
SSPA 294          NBV=KK*((NV1-LL-1)/(NBLOCK-ICOUNT)+1)
SSPA 295          IF (NBV.GT.NEQB) NBV=NEQB
SSPA 296          NBVN=0
SSPA 297    C
SSPA 298    45    IF (ICOUNT.LT.NBLOCK) GO TO 60
SSPA 299          IF (IND.EQ.1) GO TO 47
SSPA 300          KK=2*KK
SSPA 301          GO TO 90
```

```
   47 WRITE (6,1000)
      STOP
C
   50 REWIND NMASS
      REWIND NR
      DO 100 L=1,NBLOCK
      READ (NMASS) XM
      DO 120 I=1,NEQB
      VA(I,1)=XM(I)
      DO 120 J=2,NV
  120 VA(I,J)=0.0
      DO 140 K=2,NV
      II=ILQ(K-1)
      NLE=(L-1)*NEQB
      NRI=L*NEQB
      IF (II-NLE) 140,140,160
  160 IF (NRI-II) 140,180,180
  180 II=II-NLE
      VA(II,K)=1.
  140 CONTINUE
      WRITE (NR) VA
  100 CONTINUE
C
      RETURN
 1000 FORMAT (42HOWE DO NOT HAVE ENOUGH FINITE EIGENVALUES  )
      END


      SUBROUTINE REDBAK (A,VA,VV,MAXB,NEQB,NV,NWA,NWV,NWVV,NTB,NBLOCK)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      DIMENSION A(NWA),VA(NWV),VV(NWVV),MAXB(NEQB)
C
      NEB=NTB*NEQB
      NEMT=NEB+NEQB
C
C     REDUCE VECTORS ON TAPE NR
C
      REWIND NRED
      REWIND NR
      REWIND NL
      REWIND NT
      READ (NRED) A,MAXB
      ISV=NTB+1
      LL=0
      DO 10 L=1,ISV
      READ (NR) VA
      K=0
      KK=LL
      DO 20 J=1,NV
      DO 30 I=1,NEQB
      K=K+1
      KK=KK+1
   30 VV(KK)=VA(K)
   20 KK=KK+NEB
   10 LL=LL+NEQB
      ISA=1
C
  500 DO 100 I=1,NEQB
      IL=I+NEQB
      MAX=MAXB(I)
      J=0
      DO 120 II=IL,MAX,NEQB
      J=J+1
      C=A(II)
      IF (C) 110,120,110
  110 KK=I+J
      JJ=I
      DO 140 L=1,NV
      VV(KK)=VV(KK)-C*VV(JJ)
      KK=KK+NEBT
  140 JJ=JJ+NEBT
  120 CONTINUE
  100 CONTINUE
      DO 200 I=1,NEQB
      C=A(I)
      IF (C) 180,200,180
  180 KK=I
      DO 210 L=1,NV
      VV(KK)=VV(KK)/C
  210 KK=KK+NEBT
  200 CONTINUE
      IF (ISA.EQ.NBLOCK) GO TO 400
      READ (NRED) A,MAXB
      ISA=ISA+1
C
C     STORE REDUCED VECTORS ON TAPE NT
C
      K=0
      KK=0
      DO 240 J=1,NV
      DO 220 I=1,NEQB
      K=K+1
      KK=KK+1
  220 VA(K)=VV(KK)
  240 KK=KK+NEB
      WRITE (NT) VA
      K=1
      DO 310 J=1,NV
      DO 300 I=1,NEB
      KK=K+NEQB
      VV(K)=VV(KK)
  300 K=K+1
  310 K=K+NEQB
      IF (ISV.EQ.NBLOCK) GO TO 500
      READ (NR) VA
      ISV=ISV+1
      KK=NEB
      K=0
      DO 330 J=1,NV
      DO 320 I=1,NEQB
      K=K+1
      KK=KK+1
  320 VV(KK)=VA(K)
  330 KK=KK+NEB
      GO TO 500
C
C     BACKSUBSTITUTE VECTORS ON TAPE NT
C
  400 BACKSPACE NRED
      ISA=1
  420 DO 600 I=1,NEQB
```

```
      J=NEQB+1-I
      MAX=MAXB(J)
      IF (A(J)) 440,600,440
  440 KK=J
      DO 620 L=1,NV
      JJ=KK+1
      IL=J+NEQB
      C=VV(KK)
      DO 640 I=IL,MAX,NEQB
      C=C-A(I)*VV(JJ)
  640 JJ=JJ+1
      VV(KK)=C
  620 KK=KK+NFBT
  600 CONTINUE
      KK=0
      K=0
      DO 660 J=1,NV
      DO 670 I=1,NEQB
      K=K+1
      KK=KK+1
  670 VA(K)=VV(KK)
  660 KK=KK+NEB
      WRITE (NL) VA
      IF (ISA.EQ.NBLOCK) GO TO 800
      BACKSPACE NREO
      READ (NREO) A,MAXB
      BACKSPACE NREO
      ISA=ISA+1
      BACKSPACE NT
      READ (NT) VA
      BACKSPACE NT
      K=NEBT
      DO 700 J=1,NV
      DO 720 I=1,NEB
      KK=K-NEQB
      VV(K)=VV(KK)
  720 K=K-1
  700 K=K+NEBT+NEB
      K=0
      KK=0
      DO 740 J=1,NV
      DO 760 I=1,NEQB
      K=K+1
      KK=KK+1
  760 VV(KK)=VA(K)
  740 KK=KK+NEB
      GO TO 420
  800 RETURN
      END


      SUBROUTINE EIGSOL (DL,RTOLV,AR,BR,VEC,VL,VR,D,XM,NF,NV,NBLOCK,
     . NEQB,NITE)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      DIMENSION AR(NV,NV),BR(NV,NV),VEC(NV,NV),VL(NEQB,NV),VR(NEQB,NV)
      DIMENSION D(NV),DL(NV),RTOLV(NV),XM(NEQB)
C
      NITEM=12
      RTOL=1.0E-04
      TOLJ=1.0E-12
      REWIND NMASS
      REWIND NT
      REWIND NR
C
C     FIND PROJECTIONS OF MASS AND STIFFNESS OPERATORS
C
      DO 100 I=1,NV
      DO 100 J=I,NV
      AR(I,J)=0.0
  100 BR(I,J)=0.0
      DO 200 N=1,NBLOCK
      BACKSPACE NL
      READ (NL) VL
      BACKSPACE NL
      READ (NR) VR
      READ (NMASS) XM
      DO 220 I=1,NV
      DO 220 J=I,NV
      ART=0.0
      DO 230 K=1,NEQB
  230 ART=ART+VL(K,I)*VR(K,J)
  220 AR(I,J)=AR(I,J)+ART
      DO 240 I=1,NEQB
      XMM=XM(I)
      DO 240 J=1,NV
  240 VR(I,J)=VL(I,J)*XMM
      WRITE (NT) VR
      DO 260 I=1,NV
      DO 260 J=I,NV
      BRT=0.0
      DO 280 K=1,NEQB
  280 BRT=BRT+VL(K,I)*VR(K,J)
  260 BR(I,J)=BR(I,J)+BRT
  200 CONTINUE
      DO 290 I=1,NV
      DO 290 J=1,I
      AR(I,J)=AR(J,I)
  290 BR(I,J)=BR(J,I)
C
C     SOLVE EIGENVALUE PROBLEM
C
      CALL JACOBI (AR,BR,VEC,D,VL,NV,TOLJ)
      DO 295 J=1,NV
      XMM=SQRT(BR(J,J))
      DO 295 K=1,NV
  295 VEC(K,J)=VEC(K,J)/XMM
C
C     ARRANGE EIGENVALUES
C
      NV1=NV-1
  440 IS=0
      DO 400 I=1,NV1
      IF (D(I+1).GE.D(I)) GO TO 400
      IS=IS+1
      BT=BR(I+1,I+1)
      DT=D(I+1)
      BR(I+1,I+1)=BR(I,I)
      D(I+1)=D(I)
      BR(I,I)=BT
      D(I)=DT
      DO 420 K=1,NV
```

```
      TEMP=VEC(K,I+1)
      VEC(K,I+1)=VEC(K,I)
  420 VEC(K,I)=TEMP
  400 CONTINUE
      IF (IS.GT.0) GO TO 440
C
C     CHECK FOR CONVERGENCE
C
      DO 300 I=1,NV
      DIF=ABS(DL(I)-D(I))
  300 RTOLV(I)=DIF/D(I)
      WRITE (6,1040)
      WRITE (6,1000) (RTOLV(I),I=1,NV)
      DO 320 I=1,NF
      IF (RTOLV(I).GT.RTOL) GO TO 340
  320 CONTINUE
      WRITE (6,1050) RTOL
      NITE=NITEM
      GO TO 350
  340 IF (NITE.LT.NITEM) GO TO 360
      WRITE (6,1060)
  350 DO 354 I=1,NV
      DL(I)=D(I)
      IF (D(I).LE.0.0) STOP
  354 D(I)=SQRT(D(I))
      N=NT
      NT=NL
      NL=N
      N=NR
      NR=NL
      NL=N
C
      REWIND NR
      WRITE (NR) (D(I),I=1,NF)
      GO TO 430
C
C     CALCULATE APPROXIMATE EIGEN DIRECTIONS
C
  360 DO 410 I=1,NV
  410 DL(I)=D(I)
      REWIND NR
  430 REWIND NT
      DO 460 N=1,NBLOCK
      READ (NT) VR
      DO 480 J=1,NV
      DO 480 I=1,NEQB
      TEMP=0.0
      DO 500 K=1,NV
  500 TEMP=TEMP+VR(I,K)*VEC(K,J)
  480 VL(I,J)=TEMP
  460 WRITE (NR) VL
C
      RETURN
C
 1000 FORMAT (12E11.4)
 1040 FORMAT (//32H0REL TOL REACHED ON EIGENVALUES  )
 1050 FORMAT (//25H0CONVERGENCE FOR RTOL          E10.4)
 1060 FORMAT (//31H0WE ACCEPT CURRENT ITERN VALUES  )
      END

      SUBROUTINE JACOBI (A,B,X,EIGV,D,N,RTOL)
C
      DIMENSION A(N,N),B(N,N),X(N,N),EIGV(N),D(N)
C
      NSMAX=15
      DO 10 I=1,N
      D(I)=A(I,I)/B(I,I)
   10 EIGV(I)=D(I)
      IF (N.EQ.1) RETURN
      DO 30 I=1,N
      DO 20 J=1,N
   20 X(I,J)=0.
   30 X(I,I)=1.0
      NSWEEP=0
      NR=N-1
C
C     START ITERATION
C
   40 NSWEEP=NSWEEP+1
      WRITE (6,1000) NSWEEP
      EPS=(0.01**NSWEEP)**2
      DO 50 J=1,NR
      JJ=J+1
      DO 50 K=JJ,N
      TT=A(J,K)*A(J,K)
      TB=A(J,J)*A(K,K)
      EPTOLA=ABS(TT/TB)
      TT=B(J,K)*B(J,K)
      TB=B(J,J)*B(K,K)
      EPTOLB=TT/TB
      IF ((EPTOLA.LT.EPS).AND.(EPTOLB.LT.EPS)) GO TO 50
      AKK=A(K,K)*B(J,K)-B(K,K)*A(J,K)
      AJJ=A(J,J)*B(J,K)-B(J,J)*A(J,K)
      AB=A(J,J)*B(K,K)-A(K,K)*B(J,J)
      CHECK=(AB*AB+4.0*AKK*AJJ)/4.0
      IF (CHECK) 60,70,70
   60 WRITE (6,1004) CHECK
      STOP
   70 SQCH=SQRT(CHECK)
      D1=AB/2.+SQCH
      D2=AB/2.-SQCH
      DEN=D1
      IF (ABS(D2).GT.ABS(D1)) DEN=D2
      IF (DEN) 90,80,90
   80 CA=0.
      CG=-A(J,K)/A(K,K)
      GO TO 100
   90 CA=AKK/DEN
      CG=-AJJ/DEN
C
C     PERFORM GENERALIZED ROTATION
C
  100 IF (N-2) 95,180,95
   95 JP1=J+1
      JM1=J-1
      KP1=K+1
      KM1=K-1
C
      IF (JM1-1) 120,110,110
  110 DO 105 I=1,JM1
      AJ=A(I,J)
```

```
      BJ=B(I,J)
      AK=A(I,K)
      BK=B(I,K)
      A(I,J)=AJ+CG*AK
      B(I,J)=BJ+CG*BK
      A(I,K)=AK+CA*AJ
  105 B(I,K)=BK+CA*BJ
C
  120 IF (KP1-N) 130,130,140
  130 DO 125 I=KP1,N
      AJ=A(J,I)
      BJ=B(J,I)
      AK=A(K,I)
      BK=B(K,I)
      A(J,I)=AJ+CG*AK
      B(J,I)=BJ+CG*BK
      A(K,I)=AK+CA*AJ
  125 B(K,I)=BK+CA*BJ
C
  140 IF (JP1-KM1) 150,150,180
  150 DO 160 I=JP1,KM1
      AJ=A(J,I)
      BJ=B(J,I)
      AK=A(I,K)
      BK=B(I,K)
      A(J,I)=AJ+CG*AK
      B(J,I)=BJ+CG*BK
      A(I,K)=AK+CA*AJ
  160 B(I,K)=BK+CA*BJ
  180 AK=A(K,K)
      BK=B(K,K)
      A(K,K)=AK+2*CA*A(J,K)+CA*CA*A(J,J)
      B(K,K)=BK+2*CA*B(J,K)+CA*CA*B(J,J)
      A(J,J)=A(J,J)+2*CG*A(J,K)+CG*CG*AK
      B(J,J)=B(J,J)+2*CG*B(J,K)+CG*CG*BK
      A(J,K)=0.0
      B(J,K)=0.0
C
C     UPDATE EIGENVECTORS
C
      DO 190 I=1,N
      XJ=X(I,J)
      XK=X(I,K)
      X(I,J)=XJ+CG*XK
  190 X(I,K)=XK+CA*XJ
C
   50 CONTINUE
C
      DO 220 I=1,N
  220 EIGV(I)=A(I,I)/B(I,I)
      WRITE (6,1005)
      WRITE (6,1002) (EIGV(I),I=1,N)
C
C     CHECK FOR CONVERGENCE
C
      DO 240 I=1,N
      TOL=RTOL*D(I)
      DIF=ABS(EIGV(I)-D(I))
  240 IF (DIF.GT.TOL) GO TO 300
C
C     CHECK IF ALL OFF-DIAG ELEMENTS ARE SATISFACTORILY SMALL
C
      EPS=RTOL**2
      DO 260 J=1,NR
      JJ=J+1
      DO 260 K=JJ,N
      TT=A(J,K)*A(J,K)
      TB=A(J,J)*A(K,K)
      EPSA=ABS(TT/TB)
      TT=B(J,K)*B(J,K)
      TB=B(J,J)*B(K,K)
      EPSB=TT/TB
      IF ((EPSA.LT.EPS).AND.(EPSB.LT.EPS)) GO TO 260
      GO TO 300
  260 CONTINUE
C
      DO 310 I=1,N
      DO 310 J=I,N
      B(J,I)=B(I,J)
  310 A(J,I)=A(I,J)
      RETURN
C
  300 DO 320 I=1,N
  320 D(I)=EIGV(I)
      IF (NSWEEP.LT.NSMAX) GO TO 40
      DO 330 I=1,N
      DO 330 J=I,N
      B(J,I)=B(I,J)
  330 A(J,I)=A(I,J)
      RETURN
C
 1000 FORMAT (1H0,14HNO OF SWEEP =  I4)
 1002 FORMAT (     12E11.4)
 1004 FORMAT (8HOCHECK =  E20.14)
 1005 FORMAT (24HCCURRENT EIGENVALUES ARE  )
      END


      SUBROUTINE SCHECK (DL,RTOLV,A,XM,BUP,BLO,BUPC,NEIV,NWA,NEQB,
     . NBLOCK,NF,NV,SHIFT,NEI)
C
      COMMON /TAPES/NSTIF,NRED,NL,NR,NT,NMASS
      DIMENSION A(NWA),XM(NEQB),BUP(NV),BLO(NV),BUPC(NV),DL(NV),
     1RTOLV(NV)
      INTEGER NEIV(NV)
C
      RTOL=1.0E-06
      RTOL=1.0E-04
      FTOL=1.0E-02
C
      DO 100 I=1,NV
      BUP(I)=DL(I)*(1.0+FTOL)
  100 BLO(I)=DL(I)*(1.0-FTOL)
      NROOT=0
      DO 120 I=1,NF
  120 IF (RTOLV(I).LT.RTOL) NROOT=NROOT+1
      IF (NROOT.GE.1) GO TO 200
      WRITE (6,1010)
      STOP
C
C     FIND UPPER BOUNDS ON EIGENVALUE CLUSTERS
C
```

```
  200 DO 240 I=1,NROOT
  240 NEIV(I)=1
      IF (NROOT.NE.1) GO TO 260
      BUPC(1)=BUP(1)
      LM=1
      L=1
      I=2
      GO TO 295
  260 L=1
      I=2
  270 IF (BUP(I-1).LE.BLO(I)) GO TO 280
      NEIV(L)=NEIV(L)+1
      I=I+1
      IF (I.LE.NRCCT) GO TO 270
  280 BUPC(L)=BUP(I-1)
      IF (I.GT.NRCCT) GO TO 290
      L=L+1
      I=I+1
      IF (I.LE.NRCCT) GO TO 270
      BUPC(L)=BUP(I-1)
  290 LM=L
  295 IF (BUP(I-1).LE.BLO(I)) GO TO 300
      IF (RTOLV(I).GT.RTOL) GO TO 300
      BUPC(L)=BUP(I)
      NEIV(L)=NEIV(L)+1
      NROOT=NROOT+1
      IF (NROOT.EQ.NV) GO TO 300
      I=I+1
      GO TO 295
C
C     FIND SHIFT
C
  300 WRITE (6,1020)
      WRITE (6,1005) (BUPC(I),I=1,LM)
      WRITE (6,1030)
      WRITE (6,1006) (NEIV(I),I=1,LM)
      LL=LM-1
      IF (LM.EQ.1) GO TO 310
  330 DO 320 I=1,LL
  320 NEIV(L)=NEIV(L)+NEIV(I)
      L=L-1
      LL=LL-1
      IF (L.NE.1) GO TO 330
      WRITE (6,1040)
      WRITE (6,1006) (NEIV(I),I=1,LM)
  310 DO 340 I=1,LM
      IF (NEIV(I).GE.NROOT) GO TO 350
  340 CONTINUE
  350 SHIFT=BUPC(I)
      NEI=NEIV(I)
C
C     SHIFT MATRIX
C
      REWIND NSTIF
      REWIND NMASS
      REWIND NRED
      DO 400 L=1,NBLOCK
      READ (NSTIF) A
      READ (NMASS) XM
      DO 420 I=1,NEQB
  420 A(I)=A(I)-SHIFT*XM(I)
      WRITE (NRED) A
  400 CONTINUE
      I=NSTIF
      NSTIF=NRED
      NRED=I
      RETURN
C
 1005 FORMAT (1H0,6E20.12)
 1006 FORMAT (1H0,6I20)
 1010 FORMAT (30H0CONVERGENCE FOR NO EIGENVALUE  )
 1020 FORMAT (37H0UPPER BOUNDS ON EIGENVALUE CLUSTERS  )
 1030 FORMAT (34H0NO OF EIGENVALUES IN EACH CLUSTER  )
 1040 FORMAT (42H0NO OF EIGENVALUES LESS THAN UPPER BOUNDS  )
      END
```

```
      SUBROUTINE HISTRY
C
C     TIME HISTORY RESPONSE ANALYSIS TO APPLIED DYNAMIC LOADS AND/OR
C     RIGID OR MULTIPLE GROUND MOTION INPUTS
C
      COMMON/MISC/ NBLOCK,NEQB,LL,NF,LB,NSVV,NFN,NGM,NAT,NT,NOT
      COMMON/ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON/EM/ AT(1058)
      COMMON/JUNK/ NARB,NGN
      COMMON/DYN/ NTT,NOTT,DAMP,DT
      DIMENSION T(7)
      COMMON A(1)
C
C     MODE SHAPES AND FREQUENCIES
C
      CALL SECOND (T(1))
      CALL DYNAM (NEQ,MBAND,NBLOCK,NEQB,NF,NSVV,MTOT)
C
C     READ HISTORY ANALYSIS CONTROL DATA
C
      READ (5,1000) DT,DAMP
      NIT=NT
      NOTT=NOT
C
C     ADJUST NUMBER OF FREQUENCIES TO BE SMALLER THAN 5*DT
C
      N1=1
      N2=0
      TT=DT*5.
      DO 200 I=1,NF
      IF (A(I).GT.TT) N2=I
  200 CONTINUE
      NF=N2
      WRITE(6,2000) NFN,NGM,NAT,NT,NOT,DT,DAMP,NF
      NGN=NGM
      IF (NF.NE.0) GO TO 300
      WRITE(6,4000)
      STOP
  300 IF (NAT.EQ.0) NAT=1
C
C     FORM DYNAMIC LOAD VECTORS
C
      CALL SECOND (T(2))
C
C     A.  DYNAMIC FORCING FUNCTIONS
C
      N2=N1+6*NUMNP
      N3=N2+NFN*NEQB
      N4=N3+NFN*NEQB
      IF (N4.GT.MTOT) CALL ERROR (N4-MTOT)
      CALL LOAD1 (A(N1),A(N2),A(N3),NUMNP,NEQB,NFN,NEQ)
      N2=NGM-1
      IF (N2) 600,500,400
C
C     B.  ADD MULTIPLE GROUND MOTION EFFECTS
C
  400 NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL+(2*(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      N2=N1+NEQB*MBAND
      N3=N2+NEQB*LL
      N4=N3+NSBB
      CALL USOL(A(N1),A(N3),A(N4),NEQB,MBAND,LL,NBLOCK,NSB,12,3,10,11,11
     .)
C
      N2=N1+LL
      N3=N2+LL
      N4=N3+6*NUMNP
      N5=N4+NEQB
      N6=N5+NEQB*LL
      CALL MGMTN(A(N1),A(N2),A(N3),A(N4),A(N5),NEQB,LL,NUMNP,NBLOCK,NEQ)
      GO TO 600
C
C     C.  ADD RIGID GROUND MOTION EFFECTS
C
  500 N2=N1+6*NUMNP
      CALL EMID (A(N1),A(N2),NUMNP,NEQB,NEQ)
      N2=N1+NEQB*NFN
      N3=N2+NEQB*NFN
      N4=N3+NEQB
      N5=N4+NEQB
      IF (N5.GT.MTOT) CALL ERROR (N5-MTOT)
      CALL GMTN (A(N1),A(N2),A(N3),A(N4),NEQB,NFN,NBLOCK)
C
C     FORM MODAL LOAD VECTORS
C
  600 N2=N1+NFN*NF*NAT
      N3=N2+NEQB*NF
      N4=N3+NEQB*NFN
      N5=N4+NEQB*NFN
      NN6=N5+LL
      NN7=NN6+LL
      NN8=NN7+NEQB*LL
      IF (NN8.GT.MTOT) CALL ERROR (NN6-MTOT)
      N6=N2*NAT*NFN
      MAX=(MTOT-N6)/2
      N7=N6+MAX
      N8=N6+NT
      IF (N8.GT.MTOT) CALL ERROR (N8-MTOT)
      CALL LOAD2 (A(N2),A(N3),A(N4),A(N5),A(NN6),A(NN7),
     .            A(N2),A(N6),A(N7),A(N6),NEQB,NF,NFN,NT,NAR,NBLOCK,NAT,
     .            LL)
C
C     NORMAL RESPONSE
C
      CALL SECOND (T(3))
      NOS=(NT-1)/NOT
      N2=N1+NF
      N3=N2+NT
      N4=N3+NF*NOS
      IF(N4.GT.MTOT) CALL ERROR(N4-MTOT)
      CALL RESPON (A(N1),A(N2),A(N3),NF,NT,NOS)
C
C     PRINT MODE SHAPES
C
      CALL SECOND (T(4))
      N2=N1+6*NUMNP
      N3=N2+6*NF
      CALL PRINTD (A(N1),A(N2),A(N3),NEQB,NUMNP,NF,NBLOCK,NEQ,7,NF)
C
C     DISPLACEMENT RESPONSE
C
      CALL SECOND (T(5))
```

```
      NSB=NEQB*NBLOCK
      N2=N1+8*NF
      N3=N2+NF*NDS
      IF(N3.GT.MTOT) CALL ERROR(N3-MTOT)
      CALL DISPLRS(A(N1),A(N1),A(N2),A(N2),NEQB,NF,NDS,NUMNP,NBLOCK,NSB,
     .            NEQ)
C
C     STRESS RESPONSE
C
      CALL SECOND (T(6))
      N2=N1+NELTYP
      N3=N2+8*NF
      N4=N3+NSB*NF
      N5=N3+NF*NDS
      IF(N4.GT.MTOT) CALL ERROR(N4-MTOT)
      IF(N5.GT.MTOT) CALL ERROR(N5-MTOT)
      CALL STRSC1(A(N1),A(N2),A(N3),A(N3),NF,NSB,NDS,NEQB,NBLOCK)
      CALL SECOND (T(7))
      TT=0.
      DO 100 I=1,6
      T(I)=T(I+1)-T(I)
  100 TT=TT + T(I)
      T(7)=TT
      WRITE (6,3000) T
C
      RETURN
C
 1000 FORMAT (2F10.0)
 2000 FORMAT (28H1NUMBER OF DYNAMIC INPUTS..= I5//
     .          28H GROUND MOTION INDICATOR...= I5//
     .          28H NUMBER OF ARRIVAL TIMES...= I5//
     .          28H NUMBER OF TIME STEPS......= I5//
     .          28H OUTPUT INTERVAL...........= I5//
     .          28H TIME INCREMENT............= 1PE9.2//
     .          28H DAMPING FACTOR............= 1PE9.2//
     .          15H ADJUSTED NUMBER OF  /
     .          28H FREQUENCIES...............= ,I5)
 3000 FORMAT (23H1....TIME LOG (SECONDS) ///
     . 33H  MODE SHAPES AND FREQUENCIES...  ,F8.2 //
     . 33H  FORM DYNAMIC LOADS............  ,F8.2 //
     . 33H  MODAL RESPONSE................  ,F8.2 //
     . 33H  PRINT MODE SHAPES.............  ,F8.2 //
     . 33H  DISPLACEMENT OUTPUT...........  ,F8.2 //
     . 33H  STRESS OUTPUT.................  ,F8.2 //
     . 33H  TOTAL FOR RESPONSE ANALYSIS...  ,F8.2 //)
 4000 FORMAT (////50H  ALL PERIODS ARE SMALLER THAN 5*DT...STOP        )
C
      END


      SUBROUTINE LOAD1(ID,FF,IFF,NUMNP,NEQB,NFN,NFQ)
C
      DIMENSION ID(NUMNP,6),FF(NEQB,NFN),IFF(NEQB,NFN)
      COMMON / JUNK / NARB,NGM
C
C     READ ARBITRARY DYNAMIC LOADS
C
      NT=2
      REWIND NT
      REWIND 8
      READ (8) ID
C
      NNN=NEQB*NFN
      DO 10 I=1,NNN
      IFF(I)=0
   10 FF(I)=0.
C
      WRITE (6,2000)
      NARB=1
      READ (5,1000) NP,IC,IFN,IAT,P
      IF(IAT.EQ.0) IAT=1
      IF(NP.GT.0) GO TO 15
      NARB=0
      RETURN
   15 WRITE (6,2002) NP,IC,IFN,IAT,P
C
      NS=1
      NE=NEQB
      DO 500 NN=1,NUMNP
      DO 500 II=1,6
C
   20 N=ID(NN,II)
      IF (N.GT.NEQ) GO TO 500
      IF(N.LE.0) GO TO 300
      IF(N.GE.NS) GO TO 50
      WRITE (6,2001)
      STOP
C
   50 IF(N.LE.NE) GO TO 300
C
      WRITE (NT) FF,IFF
      NS=NS+NEQB
      NE=NE+NEQB
      DO 100 I=1,NNN
      IFF(I)=0
  100 FF(I)=0.
      GO TO 50
C
  300 IF(NP.EQ.NN.AND.IC.EQ.II) GO TO 350
      GO TO 500
  350 M=N-NS+1
      IF(N.LE.0) GO TO 400
      FF(M,IFN)=P
      IFF(M,IFN)=IAT
  400 READ (5,1000) NP,IC,IFN,IAT,P
      IF(IAT.EQ.0) IAT=1
      WRITE(6,2002) NP,IC,IFN,IAT,P
      GO TO 300
C
  500 CONTINUE
      WRITE (NT) FF,IFF
C
      RETURN
C
 1000 FORMAT (4I5,F10.2)
 2001 FORMAT (18HODATA OUT OF ORDER )
 2000 FORMAT (15H1DYNAMIC LOAD INPUT //
     . 57H  NODE    DISPLACEMENT  FUNCTION   ARRIVAL TIME   FUNCTION /
     . 60H  NUMBER    COMPONENT     NUMBER       NUMBER     MULTIPLIER /)
 2002 FORMAT (I6,2I11,I14,F15.3)
C
      END
```

```
      SUBROUTINE MGPTN (NFNG,NATG,ID,XM,R,NEQB,LL,NUMNP,NBLOCK,NEQ)
C
C     SET UP QUASI-STATIC INFLUENCE MATRIX FOR MULTIPLE GROUND MOTIONS
C
      DIMENSION R(NEQB,LL),NFNG(LL),NATG(LL),XM(NEQB),ID(NUMNP,6)
C
      NT=11
      REWIND 10
      REWIND 9
      REWIND 8
      READ (8) ID
      L=0
      DO 100 I=1,LL
      NFNG(I)=0
  100 NATG(I)=0
      WRITE(6,2000)
  150 L=L+1
      READ (5,1000) NP,IC,IFN,IAT
      IF (IAT.EQ.0) IAT=1
      IF (NP.LE.0) GO TO 300
      WRITE(6,2001) NP,IC,IFN,IAT
      JJ=ID(NP,IC)
      IF (JJ.GT.NEQ) GO TO 160
      WRITE(6,2002) NP,IC
      STOP
  160 JJ=JJ-NEQ
      NFNG(JJ)=IFN
      NATG(JJ)=IAT
C
      IF (L.LE.LL) GO TO 150
      WRITE(6,2003)
      STOP
C
  300 WRITE(10) NFNG,NATG
      DO 500 N=1,NBLOCK
      READ (9) XM
      BACKSPACE NT
      READ (NT) R
      BACKSPACE NT
      DO 400 I=1,NEQB
      DO 400 L=1,LL
      R(I,L)=-R(I,L)*XM(I)
  400 CONTINUE
      WRITE (10) R
  500 CONTINUE
C
      RETURN
C
 1000 FORMAT (4I5)
 2000 FORMAT (34H1MULTIPLE GROUND MOTION INPUT DATA  //
     . 46H   NODE   DISPLACEMENT  FUNCTION   ARRIVAL TIME   /
     . 46H   NO.        COMPONENT     NUMBER        NUMBER     /)
 2001 FORMAT (I6,2I11,I14)
 2002 FORMAT (//19H THE INPUT NODE NO. ,I3,22H  DISPL. COMPONENT NO.,I3,
     . 40HARE NOT COMPATIBLE WITH JOINT INPUT DATA  /
     . 24H ..EXECUTION TERMINATED.    )
 2003 FORMAT (//62H NO. OF GROUND INPUT POINTS .GT. THAT SPECIFIED BY JO
     .INT INPUT   / 24H ..EXECUTION TERMINATED.   )
C
      END


      SUBROUTINE GMTN (FF,IFF,XM,MASS,NEQB,NFN,NBLOCK)
C
      COMMON / JUNK / NARB,NGM,JFN(3),JAT(3)
      DIMENSION FF(NEQB,NFN),IFF(NEQB,NFN),MASS(NEQB),XM(NEQB)
C
C     RIGID GROUND MOTION EFFECTS
C
      JT=4
      IT=2
      REWIND IT
      REWIND JT
      REWIND 3
      REWIND 9
C
      READ (5,1000) JFN,JAT
      DO 100 I=1,3
      IF(JAT(I)) 50,50,100
   50 JAT(I)=1
  100 CONTINUE
      WRITE (6,2000) JFN,JAT
C
      NNN=NFN*NEQB
      DO 500 N=1,NBLOCK
C
      READ (3) MASS
      READ (9) XM
C
      IF(NARB.EQ.0) GO TO 200
      READ (IT) FF,IFF
      GO TO 300
  200 DO 250 I=1,NNN
      FF(I)=0.
  250 IFF(I)=0
C
  300 DO 400 I=1,NEQB
      J=MASS(I)
      IF (J.LE.0) GO TO 400
      JJ=JFN(J)
      IF(JJ.LE.0) GO TO 400
      FF(I,JJ)=-XM(I)
      IFF(I,JJ)=JAT(J)
  400 CONTINUE
C
  500 WRITE (JT) FF,IFF
C
      RETURN
C
 1000 FORMAT (6I5)
 2000 FORMAT (////30H1...GROUND MOTION INPUT KEY.../// 
     . 23X,9HDIRECTION /
     . 22X,11HX    Y    Z  //
     . 20H  FUNCTION NUMBER....,I3,2I5/
     . 20H  ARRIVAL TIME...... ,I3,2I5)
C
      END
```

```
      SUBROUTINE LOAD2 (FI,FF,IFF,NFNG,NATG,R,
     *                  PP,T,P,PD,NEQB,NF,NFN,NT,MAX,NBLOCK,NAT,LG)
C
      COMMON / EM / AT(1058)
      DIMENSION FI(NEQB,NF),FF(NEQB,NFN),IFF(NEQB,NFN),PP(NFN,1),T(1),
     *          P(1),PD(NT),NFNG(LG),NATG(LG),R(NEQB,LG)
      COMMON / DYN / QT,NOT,DAMP,DT
      COMMON / JUNK / NARB,NGM,HED(12)
      COMMON A(1)
C
C     TRANSFORM NODAL TO MODAL LOADS
C
      MTREAD=1
      IF ((NARB.EQ.0).AND.(NGM.NE.1)) MTREAD=0
      READ (5,1002) (AT(I),I=1,NAT)
      WRITE (6,2004) (I,AT(I),I=1,NAT)
      MT=2
      IF (NGM.EQ.1) MT=4
      REWIND MT
      REWIND 10
      IF (NGM.EQ.2)  READ (10) NFNG,NATG
      NE=NAT*NF*NFN
      DO 10 I=1,NE
   10 A(I)=0.
      KK=NF*NFN
C
      DO 500 N=1,NBLOCK
      BACKSPACE 7
      READ (7) FI
      BACKSPACE 7
      IF (MTREAD.NE.0) READ (MT) FF,IFF
      IF (NGM.EQ.2)  READ (10) R
      NN=-KK
C
      DO 200 I=1,NF
      DO 200 J=1,NFN
      NN=NN+1
      IF (NGM.LT.2) GO TO 100
C
      DO 80 K=1,LG
      IF (NFNG(K).NE.J) GO TO 80
      LL=NATG(K)
      IF (LL.EQ.0) GO TO 80
      DO 50 L=1,NEQB
      II=NN+LL*KK
      A(II)=A(II)+FI(L,I)*R(L,K)
   50 CONTINUE
   80 CONTINUE
C
C
  100 IF (MTREAD.EQ.0) GO TO 200
      DO 150 L=1,NEQB
      LL=IFF(L,J)
      IF(LL.EQ.0) GO TO 150
      K=NN+LL*KK
      A(K)=A(K) + FI(L,I)*FF(L,J)
  150 CONTINUE
  200 CONTINUE
  500 CONTINUE
C
C     READ TIME FUNCTIONS AND GENERATE
C     EQUAL INTERVAL FUNCTIONS
C
      DO 335 I=1,NFN
C
      READ (5,1000) NLP,SFTR,TINCR,HED
      IF(SFTR.EQ.0.) SFTR=1.0
      WRITE (6,2000) I,HED,NLP,SFTR
      IF(NLP.LE.MAX) GO TO 300
      L=2*(NLP-MAX)
      CALL ERROR(L)
  300 IF (TINCR.LE.0.) GO TO 305
      DO 303 L=1,NLP
      TL=L-1
  303 T(L)=TINCR*TL
      READ (5,1003) (P(L),L=1,NLP)
      GO TO 308
C
  305 READ (5,1001) (T(L),P(L),L=1,NLP)
C
  308 WRITE(6,2001) (T(L),P(L),L=1,NLP)
C
      TIME=T(1)
      L=1
      K=1
  310 L=L+1
      DDT=T(L)-T(L-1)
      DDP=P(L)-P(L-1)
      IF(DDT) 315,310,320
  315 WRITE (6,2003)
  320 SLOPE=DDP/DDT
  323 IF (T(L)-TIME) 310,325,325
  325 PP(I,K)=P(L-1)+(TIME-T(L-1))*SLOPE
      PP(I,K)=PP(I,K)*SFTR
  330 TIME=TIME+DT
      K=K+1
      IF (NT-K) 335,323,323
  335 CONTINUE
C
C     GENERATE MODAL LOAD VECTORS
C
      MT=4
      REWIND MT
      LL=NF*NFN
C
      DO 900 K=1,NF
      DO 550 I=1,NT
  550 PD(I)=0.
      INC=(K-1)*NFN
C
      DO 800 J=1,NAT
      LT=AT(J)/DT + 1
      N=0
C
      DO 600 NN=LT,NT
      N=N+1
      DO 600 I=1,NFN
      II=INC+I
  600 PD(NN)=PD(NN) + A(II)*PP(I,N)
C
  800 INC=INC+LL
```

```
C
  900 WRITE (MT) PC
C
      RETURN
C
 1000 FORMAT (I5,F10.0,F5.0,12A5)
 1001 FORMAT (12F6.C)
 1002 FORMAT (8F10.2)
 1003 FORMAT (1CF7.4,10X)
 2000 FORMAT (     2EH1.... TIME FUNCTION NUMBER ,I2,6X,12HHEADING ... ,
     . 12A5//6X,23HNUMBER OF LOAD POINTS = I4,/
     . 6X,23HSCALE FACTOR..........= ,F10.3//)
 2001 FORMAT (5(19H   TIME    INPUT      )/(5(F7.3,F8.3,4X)))
 2003 FORMAT (15HOBAD LOAD DATA )
 2004 FORMAT (//////14H  DELAY  TIMES //10X,TH DELAY /
     . 16H   NUMBER     TIME   / (I6,F10.2))
      END



      SUBROUTINE RESPON (W,P,X,NF,NT,NDS)
C
      DIMENSION W(NF),P(NT),X(NF,NDS)
      COMMON / DYN / MT,NOT,DAMP,DT
      COMMON / JUNK / C1,C2,C3,C4,C5,K,NOUT,F,DISP,
     .                VEL,ACEL,A,B,I,N
C
C     EVALUATION OF NORMAL RESPONSE
C
      REWIND 7
      REWIND 4
      READ (7) W
      C1=DT/2.
      C2=C1*DT/3.
      C3=C2*2.
C
      DO 260 N=1,NF
      READ (4) P
      K=1
      NOUT=NOT+1
      C4=W(N)**2
      C5=2.*DAMP*W(N)
      F=1.+C1*C5+C2*C4
      DISP=C.0
      VEL=C.0
      ACEL=F(1)
C
      DO 26C I=2,NT
      A=VEL+C1*ACEL
      B=DISP+DT*VEL+C3*ACEL
      ACEL=(P(I)-C5*A-C4*B)/F
      VEL=A+C1*ACEL
      DISP=B+C2*ACEL
      IF(NOUT-I) 26C,250,260
  250 X(N,K)=DISP
      K=K+1
      NOUT=NOUT+NOT
  260 CONTINUE
C
      REWIND 4
      WRITE (4) X
C
      RETURN
C
      END



      SUBROUTINE DISPLKS (ID,F,FI,X,NEQB,NF,NDS,NUMNP,NBLOCK,NSB,NEQ)
C
      DIMENSION ID(NUMNP,6),F(8,NF),FI(NSB ,NF),X(NF,NDS)
      COMMON / JUNK / NP,IC(6),D(8),L,  II,MSB,NS,NE,N,DDT,M,
     .                 TIME,J,K,MM,DD,KD(3,8),XUM,IEQ,KRC
     .                 ,DM(8),TP(8)
      COMMON / DYN / NT,NOT,DAMP,DT
C
C     EQN. NUMBERS CF SELECTED DISPL. COMPONENTS.
C
      REWIND 9
      REWIND 8
      READ (8) ID
      L=0
      NUM=C
      READ (5,2CC0) KKK,ISP
      WRITE (6,1005)
  100 READ (5,2CCO) NP,IC
      WRITE (6,2001) NP,IC
      IF(NP.GT.C) GC TO 110
      IF(L.EQ.0) GC TO 200
      WRITE (9) KD,L
      NUM=NUM+1
      GO TO 200
  110 DO 15C I=1,6
      II=IC(I)
      IF(II.EQ.C) GC TO 130
      IF (IC(NP,II).LE.NEQ) GO TO 120
      WRITE (6,4001) NP,II
      GO TO 150
  120 L=L+1
      KD(1,L)=NF
      KD(2,L)=II
      KD(3,L)=ID(NF,II)
      IF(ID(NP,II).LF.0) L=L-1
      IF(L.LT.8) GC TO 150
      WRITE (9) KD,L
      NUM=NUM+1
      L=0
  150 CONTINUE
      GO TO 100
C
C     APPROPRIATE MODE SHAPE COMPONENTS
C
  200 IF(NUM.EQ.C) RETURN
      WRITE (6,4CCC) KKK,ISP
      REWIND 3
      REWIND 9
      REWIND 7
      READ (7)
      NE=NSB
      NS=NE+1-NEQB
      DO 30C I=1,NBLOCK
      READ (7) ((FI(J,K),J=NS,NE),K=1,NF)
```

```
      NS=NS-NEQB
  300 NE=NE-NEQB
C
      DO 400 N=1,NUM
      READ (9) KC,L
C
      DO 350 I=1,L
      II=KD(3,I)
      DO 350 J=1,NF
  350 F(I,J)=FI(II,J)
  400 WRITE (3) L,KC,F
C
C     COMPUTE AND OUTPUT HISTORY OF VALUES
C
  410 DT=NOT*DT
C
      CALL DISPLY (X,F,NF,NDS,NUM,1,KKK,2,ISP)
C
  900 RETURN
C
 1005 FORMAT (35H1DISPLACEMENT COMPONENTS FOR WHICH /
     . 26H  TIME HISTORY IS REQUIRED  //
     . 31H  NODE  DISPLACEMENT COMPONENTS  /)
 2000 FORMAT (7I5)
 2001 FORMAT (I5,4X,6I3)
 4000 FORMAT (/18H OUTPUT TYPE.....I1/
     .        18H PLOT SPACING....I1)
 4001 FORMAT (///5H NODE,I5,25H   DISPLACEMENT COMPONENT  ,I5,6H   IS  /
     .  60H FOR GROUND MOTION INPUT....OUTPUT DISREGARD.               )
C
      END


      SUBROUTINE DISPLY (X,F,NF,NDS,NUM,NN,KKK,ISD,ISP)
C
C     S/R TO PRINT/PLOT RESPONSE VALUES
C     ISD=1..STRESS      =2..DISPL
C     KKK=1..PRINT       =2..PLOT          =3..MAXIMUMS
C
      DIMENSION X(NF,NDS),F(8,NF),NUM(NN)
      COMMON / JUNK / KD(3,8),TM(8),DM(8),D(8)
      COMMON / DYN / NT,NOT,DAMP,DT
      COMMON / FLPAR / NPAR(14)
C
      REWIND 3
      REWIND 4
      READ (4) X
C
      DO 900 N=1,NN
      REWIND 2
      REWIND 9
      MP=NUM(N)
C
      IF(ISD.EQ.2) GO TO 90
      READ (3) NPAR
      MTYPE=NPAR(1)
   90 IF(MP.EQ.0) GO TO 900
C
      DO 600 M=1,MP
      READ (3) L,KC,F
      GO TO (100,300,200) KKK
C
C     PRINT
C
  100 IF(ISD.EQ.1) GO TO 130
      WRITE (6,1000) M
      GO TO 140
  130 WRITE (6,2000) M
  140 WRITE (6,2001) (KD(1,I),KD(2,I),I=1,L)
      GO TO 300
C
C     MAXIMUMS
C
  200 IF(M.GT.1) GO TO 300
      IF(ISD.EQ.1) GO TO 230
      WRITE (6,1002)
      WRITE (6,5001)
      GO TO 300
  230 WRITE (6,2002) MTYPE
      WRITE (6,4001)
C
C     COMPUTE HISTORY
C
  300 DO 320 I=1,L
      TM(I)=0.
  320 DM(I)=0.
      TIME=0.
C
      DO 500 K=1,NDS
      TIME=TIME + DT
      DO 450 I=1,L
      DD=0.
      DO 440 J=1,NF
  440 DD = DD + F(I,J)*X(J,K)
C
      AD=ABS(DD)
      IF(AD-DM(I)) 450,450,445
  445 DM(I)=AD
      TM(I)=TIME
C
  450 D(I)=DD
      GO TO (480,450,500) KKK
C
  480 WRITE (6,1004) TIME,(D(I),I=1,L)
      GO TO 500
C
  490 WRITE (9) D
C
  500 CONTINUE
C
      GO TO (510,520,530) KKK
C
  510 WRITE (6,1005) (DM(I),I=1,L)
      WRITE (6,1006) (TM(I),I=1,L)
      GO TO 600
C
  520 WRITE (2) KC,DM,TM,L
      GO TO 600
C
  530 WRITE (6,1007) (KD(1,I),KD(2,I),DM(I),TM(I),I=1,L)
C
  600 CONTINUE
```

```
C
C     PLOT SET OF VALUES
C
      IF(KKK.NE.2) GO TO 900
      REWIND 2
      REWIND 9
      DO 800 M=1,MM
      GO TO (610,620) ISD
C
  610 WRITE (6,4000) MTYPE,M
      WRITE (6,4001)
      GO TO 630
C
  620 WRITE (6,5000) M
      WRITE (6,5001)
C
  630 CALL PLOT (2,S,NDS,ISP)
C
  800 CONTINUE
  900 CONTINUE
C
C
      RETURN
C
 1000 FORMAT (30H1TIME HISTORY FOR SELECTED DISPLACEMENT COMPONENTS ,
     . 5H.... ,I3//10X,40HNODE NUMBERS AND DISPLACEMENT COMPONENTS )
 1002 FORMAT (55H1MAXIMUM DISPLACEMENT VALUES FROM DYNAMIC RESPONSE ANAL
     .YSIS //)
 1004 FORMAT (F8.3,2X,1P8E12.3)
 1005 FORMAT ( /24H MAXIMUM ABSOLUTE VALUES //
     . 10H MAXIMUM   ,1P8E12.3)
 1006 FORMAT (10H TIME       ,1P8E12.3)
 1007 FORMAT (I6,I13,1PE18.3,E12.3,5X,2HN4)
 2000 FORMAT (  44H1TIME HISTORY FOR SELECTED STRESS COMPONENTS ,
     . 5H.... , I3//10X,37H ELEMENT AND STRESS COMPONENT NUMBERS )
 2001 FORMAT (8H    TIME , 2X, 8(I8,1H-,I2,X))
 2002 FORMAT (16H1ELEMENT TYPE... ,I2//
     . 53H MAXIMUM STRESS VALUES FROM DYNAMIC RESPONSE ANALYSIS //)
 4000 FORMAT (17H1ELEMENT TYPE....,I2//
     . 39H NORMALISED PLOT OF STRESS HISTORIES... ,I3/)
 4001 FORMAT (58H  ELEMENT       STRESS        MAXIMUM       TIME AT      PL
     .OT / 58H  NUMBER     COMPONENT        VALUE       MAXIMUM     SYMBOL
     .)
 5000 FORMAT (46H1NORMALISED PLOT OF DISPLACEMENT HISTORIES.....I3/)
 5001 FORMAT(58H    NODE     DISPLACEMENT     MAXIMUM        TIME AT     PLO
     .T / 58H   NUMBER      COMPONENT          VALUE        MAXIMUM    SYMBOL)
C
      END


      SUBROUTINE STRSDI (NUM,SF,FI,X,NF,NSB,NDS,NEQB,NBLOCK)
C
      DIMENSION NUM(1),SF(8,NF),FI(NSB,NF),X(NF,NDS)
      COMMON / EM / NS,ND,LM(24),SA(12,24),EXRA(744)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON / JUNK / N,NEL,IS(12),M,I,L,KS(3,8),II,K,J,SS,JJ
     .                  ,NUME,NE
C
C     ASSEMBLE MODE SHAPES IN CORE
C
      REWIND 7
      READ (7)
      NE=NSB
      NS=NE+1-NECB
      DO 100 I=1,NBLOCK
      READ (7) ((FI(J,K),J=NS,NE),K=1,NF)
      NS=NS-NEQB
  100 NE=NE-NEQB
C
C     FORM STRESS MATRIX,MODE SHAPE TRANSFORMATION FOR
C     SELECTED STRESS COMPONENTS ONLY.
C
      REWIND 1
      REWIND 3
C
      READ (5,1000) KKK,ISP
      WRITE (6,3000)
      DO 500 N=1,NELTYP
      READ (1) NPAR
      WRITE (3) NPAR
      WRITE (6,3001) NPAR(1)
      READ (5,1000) NEL,IS
      WRITE (6,2000) NEL,IS
      NUME=NPAR(2)
      L=0
      NUM(N)=0
C
      DO 400 M=1,NUME
      READ (1) NS,ND,LM,SA
      IF(NEL.NE.M) GO TO 400
C
      DO 300 I=1,NS
      II=IS(I)
      IF(II.EQ.0) GO TO 350
      L=L+1
      KS(1,L)=NEL
      KS(2,L)=II
C
      DO 200 K=1,NF
      SS=0.
      DO 150 J=1,ND
      JJ=LM(J)
      IF (JJ.GT.NEQ) GO TO 150
      IF(JJ) 150,150,140
  140 SS = SS + SA(II,J)*FI(JJ,K)
  150 CONTINUE
  200 SF(L,K)=SS
C
      IF(L.LT.8) GO TO 300
      WRITE (3) L,KS,SF
      L=0
      NUM(N)=NUM(N) + 1
  300 CONTINUE
  350 READ  (5,1000) NEL,IS
      WRITE (6,2000) NEL,IS
  400 CONTINUE
C
      IF(L.EQ.0) GO TO 500
      WRITE (3) L,KS,SF
      NUM(N)=NUM(N) + 1
  500 CONTINUE
      WRITE (6,4000) KKK,ISP
```

```
C
C     COMPUTE AND OUTPUT HISTORY OF VALUES
C
      CALL DISPLY (X,SF,NF,NDS,NUM,NELTYP,KKK,1,ISP)
C
      RETURN
C
 1000 FORMAT (1315)
 2000 FORMAT (I6,4X,12I3)
 3000 FORMAT (28H1STRESS COMPONENTS FOR WHICH /
     . 25H TIME HISTORY IS REQUIRED )
 3001 FORMAT (/16H ELEMENT TYPE .. ,I2//
     . 38H ELEMENT      DESIRED STRESS COMPONENTS )
 4000 FORMAT (/16H OUTPUT TYPE.....I1/
     .         16H PLOT SPACING....I1)
C
      END



      SUBROUTINE PLOT (IT,JT,NDS,ISP)
C
      COMMON /  FM  / PP(101),KD(3,8),XM(8),TM(8),IP(8),X(8)
      DIMENSION SM(8)
      DATA SM / 1H1,1H2,1H3,1H4,1H5,1H6,1H7,1H8 /
      DATA BL,V /1H ,1HX /
      COMMON / DYN / NT,NDI,DAMP,DT
C
C
      READ (IT) KD,XM,TM,L
      WRITE (6,3000) (KD(1,I),KD(2,I),XM(I),TM(I),I,I=1,L)
C
      DO 100 I=1,L
      IF(XM(I)) 50,100,50
   50 XM(I)=50./XM(I)
  100 CONTINUE
      TT=0.
      WRITE (6,999)
      WRITE (6,1000)
      WRITE (6,2000) TT,(V,I=1,101),TT
C
      K=1
      DO 200 I=2,100
  200 PP(I)=BL
C
      DO 500 N=1,NDS
      READ (JT) X
      PP(1)=V
      PP(51)=V
      PP(101)=V
C
  220 II=ISP
  210 IF(II.LE.0) GO TO 250
      WRITE (6,2001) PP
      II=II-1
      GO TO 210
C
  250 TT=TT+DT
      DO 300 I=1,L
      XX=XM(I)*X(I)
      M=XX
      M=M+51
      IP(I)=M
  300 PP(M)=SM(I)
      IF(K.LT.10)  GO TO 320
      K=1
      WRITE (6,2000) TT,PP,TT
      GO TO 340
  320 WRITE (6,2001) PP
      K=K+1
C
C     RESET PP
C
  340 DO 360 I=1,L
      M=IP(I)
  360 PP(M)=BL
  500 CONTINUE
      TT=TT+DT
      WRITE (6,2000) TT,(V,I=1,101),TT
      WRITE (6,1000)
C
      RETURN
C
C     CARRIAGE CONTROL CHARACTER Z SUPRESSES
C     SKIP OVER PAGE FOLDS
C
  999 FORMAT (//8X,8HORDINATE)
 1000 FORMAT (/12HZ  TIME -1.0,21X,4H 0.5,22X,3H0.0,22X,3H0.5,22X,
     . 3H1.0, X,4HTIME)
 2000 FORMAT (1HZ,F7.2,2X,101A1,F7.2)
 2001 FORMAT (1HZ,9X,101A1)
 3000 FORMAT (I6,I12,1PE18.3,E12.3,I6)
      END
```

```
      SUBROUTINE EMID(ID,MASS,NUMNP,NEQB,NEQ)
      DIMENSION ID(NUMNP,6),MASS(NEQB)
C
      REWIND 3
      REWIND 8
      READ (8) ID
      L=1
      DO 200 N=1,NUMNP
      DO 100 I=1,6
   50 MASS(L)=0
      II=ID(N,I)
      IF (((II.LE.0).OR.(II.GT.NEQ)) GO TO 100
      IF(L.LE.NEQB) GO TO 75
      WRITE (3) MASS
      L=1
   75 IF(I.GT.3) GO TO 90
      MASS(L)=I
   90 L=L+1
  100 CONTINUE
  200 CONTINUE
      DO 300 I=L,NEQB
  300 MASS(I)=0
      WRITE (3) MASS
C
      RETURN
C
      END
```

```
      SUBROUTINE RESPEC
C
C
      COMMON / MISC / NBLOCK,NEQB,LL,NF,LB,NSVV
      COMMON /JUNK/ XXX(4),NDYN,JUK(200)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      DIMENSION T(6)
      COMMON A(1)
C
C
      CALL SECOND (T(1))
C
      IF (NSVV.LT.10) NSVV=10
      CALL DYNAM (NEQ,MBAND,NBLOCK,NEQB,NF,NSVV,MTOT)
C
      CALL SECOND (T(2))
C
       N1=1
      N2=N1 + 6*NUMNP
      CALL EMID (A(N1),A(N2),NUMNP,NEQB,NEQ)
      N2=N1+NEQB*NF
      N3=N2+NF*2
      N4=N3+NEQB
      N5=N4+NF
      N6=N5+NEQB
      N7=N6+NF
      IF(N7.GT.MTOT) CALL ERROR(N7-MTOT)
      CALL SPECTRM (A(N1), A(N2),A(N3),A(N4),A(N5),NEQB,NF,NBLOCK,A(N6))
C
C     MODE SHAPE NG IS R.M.S. DISPLACEMENT
C
      CALL SECOND (T(3))
      N2=N1+6*NUMNP
      NG=NF+1
      N3=N2+6*NG
      N4=N3+NEQB*NG
      MM=N4-MTOT
      IF (MM.GT.0) CALL ERROR (MM)
      NT=2
      CALL PRINTD(A(N1),A(N2),A(N3),NEQB,NUMNP,NG,NBLOCK,NEQ,NT,NF)
C
C     COMPUTE STRESSES
C
      CALL SECOND (T(4))
      N2=N1+4*LL
      N3=N2+NEQB*LL
      LB=(MTOT-N3)/(NEQ +12)
      LL=NF
      NDYN=2
      CALL STRESS (A(N1),A(N2),A(N3),NEQB,LB,LL,NEQ,NBLOCK)
C
C     R M S COMBINATION OF STRESSES
C
      CALL SECOND (T(5))
      CALL RMS
      CALL SECOND (T(6))
C
      TT=0.
      DO 100 I=1,5
      T(I)=T(I+1)-T(I)
  100 TT=TT + T(I)
```

```
      T(6)=TT
      WRITE (6,1000) T
C
      RETURN
C
 1000 FORMAT (23H1....TIME LOG (SECONDS)  ///
     . 33H  MODE SHAPES AND FREQUENCIES...   ,F8.2//
     . 33H  MAXIMUM MODAL DISPLACEMENTS...   ,F8.2//
     . 33H  PRINT MODE SHAPES.............   ,F8.2//
     . 33H  COMPUT MODAL STRESSES.........   ,F8.2//
     . 33H  R.M.S. COMBINATION OF STRESSES   ,F8.2//
     . 33H  TOTAL FOR SPECTRUM ANALYSIS...   ,F8.2//)
      END


      SUBROUTINE SPECFRM (F,PX,XM,W,MASS,NEQB,NF,NBLOCK,TM)
      DIMENSION PX(NF,3),F(NEQB,NF),XM(NEQB),W(NF),MASS(NEQB)
      DIMENSION DIRN(3),TM(NF)
      DATA DIRN /3H X ,3H Y ,3H Z /
C
C     COMPUTES MODAL AND R.M.S. DISPL RESPONSE TO EARTHQUAKE
C
      TPI=6.2831853
      DO 100 I=1,NF
      TM(I)=0.
      DO 100 J=1,3
  100 PX(I,J)=0.
C
C      FORM MODAL PARTICIPATION FACTORS PX(I,IDRN)
C           IDRN=1,2,3 .... FOR X,Y,Z, DIRN EARTHQUAKE
C
      REWIND 9
      REWIND 3
      REWIND 7
      READ (7)
      DO 5 I=1,NBLOCK
    5 READ (7)
      DO 200 N=1,NBLOCK
      BACKSPACE 7
      READ (7) F
      BACKSPACE 7
      READ (3) MASS
      READ (9) XM
C
      DO 250 I=1,NEQB
      J=MASS(I)
      IF (J.LE.0) GO TO 250
      DO 240 L=1,NF
      TM(L)=TM(L) + F(I,L)*F(I,L)*XM(I)
  240 PX(L,J)=PX(L,J)+F(I,L)*XM(I)
  250 CONTINUE
  200 CONTINUE
      CALL FRMAT (PX,NF,3,NF,2HPX)
      CALL FRMAT (TM,NF,1,NF,2HTM)
C
C     READ FREQUENCIES W OFF TAPE 7
C
      BACKSPACE 7
      READ (7) W
      REWIND 2
      WRITE (2) W
C
C     COMPUTE MODAL AMPLITUDES (IN W) FROM SPECTRUM AND PX
C
      READ (5,1000) IDRN
      WRITE (6,2000) DIRN(IDRN)
      DO 300 I=1,NF
      WW=TPI/W(I)
  300 W(I)=SD(WW)*PX(I,IDRN)/TM(I)
      CALL FRMAT (W,NF,1,NF,3HAMP )
C
C     WRITE MODAL DISPLS F AND R.M.S. ON TAPE 2
C
      REWIND 7
      READ (7)
      DO 350 N=1,NBLOCK
      READ (7) F
      DO 310 J=1,NF
      AMP=W(J)
      DO 310 I=1,NEQB
  310 F(I,J)=F(I,J)*AMP
C
      DO 320 I=1,NEQB
      WW=0.
      DO 330 J=1,NF
  330 WW=WW+F(I,J)**2
  320 XM(I)=SQRT(WW)
  350 WRITE (2) F,XM
C
      RETURN
 1000 FORMAT (I5)
 2000 FORMAT (28H1 RESPONSE SPECTRUM ANALYSIS  //
     .  13H  CONSIDERING ,A3,9HDIRECTION  )
      END


      FUNCTION   SD(TT)
C
      COMMON / JUNK / MM,L,K,NTAG,NDYN,T(90),S(90),HED(12),T,W,SS,ST,
      IF (NTAG.EQ.1) GO TO 500
      NTAG=1
C
C     READ ACCELERATION SPECTRUM
C
      READ  (5,1000) HED,NPTS,SFTR,(T(I),S(I),I=1,NPTS)
      WRITE (6,2000) HED,NPTS,SFTR,(T(I),S(I),I=1,NPTS)
      TPI=8.*ATAN(1.)
  500 CONTINUE
C
      DO 600 I=1,NPTS
      IF (TT.LT.T(I)) GO TO 700
  600 CONTINUE
  700 TI=T(I)-T(I-1)
      SI=S(I)-S(I-1)
      SS=S(I-1)+SI*(TT-T(I-1))/TI
      W=TPI/TT
      SD=SFTR*SS/W**2
C
 1000 FORMAT (12A6/I5,F10.2/(2F10.2))
 2000 FORMAT (1H0,12A6//I6,F15.3//(1P2E14.2))
```

```
C
      RETURN
      END
      SUBROUTINE RMS
C
C     COMPUTE AND PRINT RMS STRESSES
C
      COMMON / MISC / NBLOCK,NEQB,LL,NF,LB
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON / JUNK / N,I,K,NTAG,NDYN,S(12),SIG(12)
C
      I1=1
      I2=2
      REWIND 3
C
      DO 700 MM=1,NELTYP
      WRITE (6,1000)
      REWIND I1
      REWIND I2
      READ (3) NPAR
      MTYPE=NPAR(1)
      NUMEL=NPAR(2)
      NPAR(1)=0
      NTAG=0
C
      DO 500 L=1,NF,LB
      DO 400 N=1,NUMEL
      DO 20 I=1,12
   20 S(I)=0.
      IF (L.NE.1) READ (I2) S
      KK=NF-L-LB
      IF (KK.LT.0) LB=KK+1+LB
      DO 100 K=1,LB
      READ (3) NS,SIG
      DO 50 I=1,NS
   50 S(I)=S(I)+SIG(I)**2
  100 CONTINUE
C
      IF (KK.GE.0) GO TO 350
C
C     PRINT RMS STRESSES
C
      DO 60 I=1,NS
   60 S(I)=SQRT(S(I))
      I=0
      GO TO (1,2,3,4,5) MTYPE
C
    1 CALL TRUSS
      GO TO 400
C
    2 CALL BEAM
      GO TO 400
C
    3 CALL CBEAM
      GO TO 400
C
    4 CALL BOUND
      GO TO 400
C
    5 CALL EXPJP
      GO TO 400
C
  350 WRITE (I1) S
C
  400 CONTINUE
      REWIND I1
      REWIND I2
      I3=I1
      I1=I2
  500 I2=I3
C
  700 CONTINUE
C
      RETURN
 1000 FORMAT (28H1 RESPONSE SPECTRUM ANALYSIS   //
     .  27H  ROOT MEAN SQUARE STRESSES  )
      END
```

```
      SUBROUTINE TRUSS
C
      COMMON A(1)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON / JUNK / MM,L,K,NTAG,NDYN,SIG(12),EXPA(188)
C
      IF(NPAR(1).EQ.0) GO TO 500
      N6=N5+NPAR(3)
      N7=N6+NPAR(3)
      N8=N7+NPAR(3)
      N9=N8+NPAR(3)
      MM=N9+NPAR(3)-MTOT
      IF(MM.GT.0) CALL ERROR(MM)
      CALL RUSS(A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9),
     1 NUMNP)
      RETURN
C
  500 IF (NTAG.EQ.0) WRITE (6,2002)
  352 WRITE(6,3002) MM,L,SIG(1),SIG(2)
      NTAG=1
C
      RETURN
 2002 FORMAT (//23H    TRUSS MEMBER ACTIONS //
     *           40H0 MEMBER     LOAD           STRESS            FORCE  )
 3002 FORMAT (2I8,F15.5,F15.3)
      END


      SUBROUTINE RUSS(ID,X,Y,Z,E,THERM,DEN,AREA,WT,NUMNP)
C
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1),E(1),THERM(1),DEN(1),AREA(1)
     1 ,WT(1)
      COMMON /ELPAR/ NPAR(14),NNNNN,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON /EM/LM(24),ND,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
      COMMON /JUNK/ EMUL(4,4),I,J,K,L,M,N,II,JJ,KK,MTYPE,TEMP,DX,DY,DZ,
     1 XL2,XL,XX,YY,F,FT,FX,FY,FZ,MIN,MAX,NDIF,KKK,TEM,MTYP
C
C     CONTROL INFORMATION AND MEMBER PROPERTIES
C
      NUME=NPAR(2)
      NUMMAT=NPAR(3)
      WRITE (6,2000) NUME,NUMMAT
      WRITE (6,2001)
      DO 10 I=1,NUMMAT
      READ  (5,1001) N,E(N),THERM(N),DEN(N),AREA(N),WT(N)
   10 WRITE (6,2002) N,E(N),THERM(N),DEN(N),AREA(N),WT(N)
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ (5,1003) EMUL
      WRITE (6,2003) EMUL
C
C     ELEMENT INFORMATION
      WRITE (6,2005)
C
      N=1
  100 READ (5,1004) M,II,JJ,MTYP,TEM,KK
      IF(KK.EQ.0) KK=1
  120 IF(M.NE.N) GO TO 200
      I=II
      J=JJ
      MTYPE=MTYP
      TEMP=TEM
      KKK=KK
C
C     1. FORM ELEMENT STIFFNESS AND STRESS MATRICES
C
  200 DX=X(I)-X(J)
      DY=Y(I)-Y(J)
      DZ=Z(I)-Z(J)
      XL2=DX*DX+DY*DY+DZ*DZ
      XL=SQRT(XL2)
      XX=E(MTYPE)*AREA(MTYPE)*XL
      ST(1,1)=DX/XL2
      ST(1,2)=DY/XL2
      ST(1,3)=DZ/XL2
      ST(1,4)=-ST(1,1)
      ST(1,5)=-ST(1,2)
      ST(1,6)=-ST(1,3)
C
      DO 300 L=1,6
      YY=ST(1,L)*XX
      DO 250 K=L,6
      S(K,L)=ST(1,K)*YY
  250 S(L,K)=S(K,L)
      ST(1,L)=E(MTYPE)*ST(1,L)
  300 ST(2,L)=AREA(MTYPE)*ST(1,L)
C
C     2. INERTIA AND THERMAL LOADS
C
      F=WT(MTYPE)*XL/2.
      FT=TEMP*THERM(MTYPE)*E(MTYPE)*AREA(MTYPE)
      FX=DX*FT/XL
      FY=DY*FT/XL
      FZ=DZ*FT/XL
C
      DO 350 L=1,4
      TT(2,L)=EMUL(L,4)*FT
      TT(1,L)=TT(2,L)/AREA(MTYPE)
      P(1,L)=EMUL(L,1)*F-EMUL(L,4)*FX
      P(2,L)=EMUL(L,2)*F-EMUL(L,4)*FY
      P(3,L)=EMUL(L,3)*F-EMUL(L,4)*FZ
      P(4,L)=EMUL(L,1)*F+EMUL(L,4)*FX
      P(5,L)=EMUL(L,2)*F+EMUL(L,4)*FY
  350 P(6,L)=EMUL(L,3)*F+EMUL(L,4)*FZ
      F=DEN(MTYPE)*AREA(MTYPE)*XL/2.
      DO 375 L=1,6
  375 XM(L)=F
C
C     3. FORM LOCATION MATRIX AND COMPUTE BAND WIDTH
C
      DO 400 L=1,3
      LM(L)=ID(I,L)
  400 LM(L+3)=ID(J,L)
C
      ND=6
      NS=2
      CALL WRITET(MBAND,NDIF)
C
C     4. CHECK FOR MORE ELEMENTS
C
```

```
      WRITE (6,2004) N,I,J,MTYPE,TEMP,NDIF
      IF (N.EQ.NUME) RETURN
      N=N+1
      I=I+KKK
      J=J+KKK
      IF(N.GT.M) GC TO 100
      GC TC 120
C
 1001 FORMAT (I5,5F10.0)
 1003 FORMAT (4F10.C)
 1004 FORMAT (4I5,1F10.0,I5)
 2000 FORMAT (///25HINUMBER OF TRUSS MEMBERS= I5/
     1 25H NUMBER CF DIFF. MEMBERS= I5)
 2001 FORMAT (///1X,4HTYPE,14X,1HE,10X,5HALPHA,12X,3HDEN,11X,4HAREA
     1 11X,4HWT/L )
 2002 FORMAT (I5,5E15.7)
 2003 FORMAT(///25H ELEMENT LOAD MULTIPLIERS / 20X,1HA,14X,1HB,14X,1HC,
     1 14X,1HD,/6H X-DIR4E15.6/ 6H Y-DIR4E15.6/ 6H Z-DIR4E15.6/
     2 6H   TEMP4E15.6)
 2004 FORMAT (4I6,F10.2,I7)
 2005 FORMAT (///42H1    N     I     J  TYPE      TEMP   BAND  )
      END
```

```
      SUBROUTINE BEAM
C
      COMMON A(1)
      CCMMCN /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTCT,NEQ
      COMMON / JUNK / MM,L,K,NTAG,NDYN,SIG(12),EXRA(188)
C
      IF(NPAR(1).EC.0) GO TO 500
      N6=N5+NPAR(5)
      N7=N6+NPAR(5)
      N8=N7+NPAR(5)
      N9=N8+12*NPAR(4)
      N10=N9+6*NPAR(3)
      IF(N10.GT.MTCT) CALL ERROR(N10-MTOT)
      CALL TEAM(NPAR(2),NPAR(3),NPAR(4),NPAR(5),A(N1),A(N2),A(N3),
     .          A(N4),A(N5),A(N6),A(N7),A(N8),A(N9),NUMNP,MBAND)
      RETURN
C
  500 IF(NTAG.EC.0) WRITE (6,2002)
      WRITE(6,3C02) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
 2002 FORMAT(/29H0.....BEAM FORCES AND MOMENTS//
     . 10H0BEAM LOAD 5X 5HAXIAL  2(7X,5HSHEAR),5X 7HTORSION
     .  2(5X,7HBENDING)/ 10H  NO.  NO.  8X  2HR1  10X  2HR2  10X
     .  2HR3 10X 2HM1 10X 2HM2 10X 2HM3)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
      END
```

```
      SUBROUTINE TEAM(NBEAM,NUMETP,NUMFIX,NUMMAT,ID,X,Y,Z,E,G,RC,
     .SFT, COPRCP,NUMNP,MBAND)
C
C     FORMS 3-D BEAM STIFFNESS AND STRESS ARRAYS
C
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .    SF(12,4)
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1),E(1),G(1),SFT(NUMFIX,1)
     . ,COPROP(NUMETP,1),RO(1),EMUL(3,4)
      COMMON /NEWB/ LC(4),T(3,3),JK(6),MELTYP,DL,MATTYP
      DIMENSION ILC(4),TI(3,3),TJ(3,3),STIF(722),TS(2,2),LS(4)
      EQUIVALENCE (STIF,LM)
C
C     INITIALIZATION
C
      WRITE (6,2005) NBEAM,NUMETP,NUMFIX,NUMMAT
      N=0
      DO 5 I=1,1056
    5 STIF(I)=0.
C
C     READ AND PRINT MATERIAL PROPERTY DATA
C
      WRITE (6,2001)
      DO 10 I=1,NUMMAT
      READ  (5,1001) N,E(N),G(N),RO(N)
      WRITE (6,2002) N,E(N),G(N),RO(N)
   10 G(N)=0.5*E(N)/(1.+G(N))
C
C     READ AND PRINT GEOMETRIC PROPERTIES OF COMMON ELEMENTS.
C
```

```
      WRITE (6,2003)
      DO 30 I=1,NUMETP
      READ  (5,1002) N,(COPROP(N,J),J=1,6)
      IF((COPROP(N,1).NE.0.0).AND.(COPROP(N,4).NE.0.0).AND.
     1   (COPROP(N,5).NE.0.0).AND.(COPROP(N,6).NE.0.0))  GO TO 20
      WRITE (6,2013)
      CALL EXIT
   20 WRITE (6,2004) N,(COPROP(N,J),J=1,6)
   30 CONTINUE
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ  (5,1006) ((EMUL(I,J),J=1,4),I=1,3)
      WRITE (6,2006) ((EMUL(I,J),J=1,4),I=1,3)
C
C     READ AND PRINT FIXED END FORCES IN LOCAL COORDINATES
C
      IF(NUMFIX .EQ. 0) GO TO 56
      WRITE (6,2010)
      DO 55 I=1,NUMFIX
      READ  (5,1005) N,(SET(N,J),J=1,12)
   55 WRITE (6,2011) N,(SET(N,J),J=1,12)
   56 CONTINUE
C
C     READ AND PRINT ELEMENT DATA. GENERATE MISSING INPUT.
C
      WRITE (6,4000)
      L=0
   60 KKK=0
      READ (5,3000) INEL,INI,INJ,INK,IMAT,IMEL,ILC,INELKI,INELKJ,INC
      IF (INEL.NE.1) GO TO 15
      NI=INI
      NJ=INJ
      NK=INK
   15 IF (INC.LE.0) INC=1
   65 L=L+1
      KKK=KKK+1
      ML=INEL-L
      IF (ML)  66,67,68
   66 WRITE (6,4003) INEL
      CALL EXIT
   67 NEL=INEL
      NI=INI
      NJ=INJ
      NK=INK
      MATTYP=IMAT
      MELTYP=IMEL
      DO 90 I=1,4
   90 LC(I)=ILC(I)
      NEKODI=INELKI
      NEKODJ=INELKJ
      DO 91 I=1,3
   91 T(2,I)=TI(2,I)
      GO TO 69
   68 NEL=INEL-ML
      NI =INI+KKK*INCR
      NJ =JN+KKK*INCP
   69 CONTINUE
      WRITE (6,4001) NEL,NI,NJ,NK,MATTYP,MELTYP,LC,NEKODI,NEKODJ
C
   74 DX=X(NJ)-X(NI)
      DY=Y(NJ)-Y(NI)
      DZ=Z(NJ)-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF(DL) 75,75,76
   75 WRITE (6,4005) NEL
      CALL EXIT
C
C     FORM GLOBAL TO LOCAL COORDINATE TRANSFORMATION.
C
   76 T(1,1)=DX/DL
      T(1,2)=DY/DL
      T(1,3)=DZ/DL
C
C     COMPUTE DIRECTION COSINES OF LOCAL Y-AXIS
C
      A1=X(NJ)-X(NI)
      A2=Y(NJ)-Y(NI)
      A3=Z(NJ)-Z(NI)
      B1=X(NK)-X(NI)
      B2=Y(NK)-Y(NI)
      B3=Z(NK)-Z(NI)
      AA=A1*A1+A2*A2+A3*A3
      AB=A1*B1+A2*B2+A3*B3
      U1=AA*B1-AB*A1
      U2=AA*B2-AB*A2
      U3=AA*B3-AB*A3
      UU=U1*U1+U2*U2+U3*U3
      UU=SQRT(UU)
      IF (UU.GT.0.) GO TO 40
      WRITE (6,4002) INEL
      STOP
   40 T(2,1)=U1/UU
      T(2,2)=U2/UU
      T(2,3)=U3/UU
      T(3,1)=T(1,2)*T(2,3)-T(1,3)*T(2,2)
      T(3,2)=T(1,3)*T(2,1)-T(1,1)*T(2,3)
      T(3,3)=T(1,1)*T(2,2)-T(1,2)*T(2,1)
C
C     CHECK IF NEW STIFFNESS NEEDED
C
      IF (NEL.GE.1) GO TO 80
      IF  (ABS(DS-DL).GT.DL/100.)  GO TO 80
      IF ((MT.NE.MATTYP).OR.(ME.NE.MELTYP)) GO TO 80
      IF ((JK(1).NE.NEKODI).OR.(JK(2).NE.NEKODJ)) GO TO 80
      DO 81 I=1,4
      IF (LS(I).NE.LC(I)) GO TO 80
   81 CONTINUE
      DO 82 I=1,2
      DO 82 J=1,2
      IF (ABS(TS(I,J)-T(I,J)).GT.ABS(T(I,J)/100.)) GO TO 80
   82 CONTINUE
      GO TO 185
C
   80 DS=DL
      MT=MATTYP
      ME=MELTYP
      DO 77 I=1,2
      DO 77 J=1,2
   77 TS(I,J)=T(I,J)
      DO 78 I=1,4
   78 LS(I)=LC(I)
      JK(1)=NEKODI
      JK(2)=NEKODJ
```

```
C
C     FORM NEW STIFFNESS
C
      CALL NEWBM (E,G,RO,COPROP,SFT,NUMFIX,NUMETP)
C
C     ADD GRAVITY LOADING ... POINT LOADS ONLY COMPUTED
C
      DO 180 I=1,3
      DO 18C J=1,4
      RF(I,J)=RF(I,J)+EMUL(I,J)*XM(I)
  180 RF(I+6,J)=RF(I+6,J)+EMUL(I,J)*XP(I+6)
C
C     FORM ELEMENT LOCATION MATRIX
C
  185 CONTINUE
      DO 17C M=1,6
      LM(M)=ID(NI,M)
      LM(M+12)=0
      LM(M+18)=C
  170 LM(M+6)=ID(NJ,M)
C
      NS=12
      ND=12
C
C     TRANSFORM TO MASTER DEGREES OF FREEDOM
C
      CALL SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITET (MBAND,NDIF)
C
C     CHECK FOR LAST ELEMENT
C
      IF (NBEAM-NEL) 66,500,260
  260 IF (ML.GT.0)  GO TO 65
      IN=INI
      JN=INJ
      INCR=INC
      GO TO 60
  5C0 RETURN
C
 1001 FORMAT(I5,3F1C.0)
 1002 FORMAT(I5,6F1C.0)
 1005 FORMAT(I5,6F1C.0/F15.0,5F1C.0)
 1006 FORMAT (4F10.0)
 2001 FORMAT(55H1MATERIAL    YOUNG S      POISSON S              MASS
     1      /55H             MODULUS       RATIO              DENSITY       )
 2002 FORMAT(1H ,I5,3X,F12.0,F14.5,F14.5)
 2003 FORMAT(1H1/
     .  26H BEAM GEOMETRIC PROPERTIES//
     1    48H ELEMENT        AREA          AREA              AREA
     2  60H INERTIA       INERTIA       INERTIA
     3 / 48H  TYPE           X              Y              Z
     4                    30H    X              Y              Z                 )
 2004 FORMAT(1H ,I5,2X,6F12.3)
 2005 FORMAT(39H1.......THREE DIMENSIONAL BEAM ELEMENTS//
     .   36H  NUMBER OF BEAMS                     =,I5/
     .   36H  NUMBER OF GEOMETRIC PROPERTY SETS=,I5/
     .   36H  NUMBER OF FIXED END FORCE SETS    =,I5/
     .   36H  NUMBER OF MATERIALS               =,I5)
 2006 FORMAT(///25H ELEMENT LOAD MULTIPLIERS / 20X,1HA,14X,1HB,14X,1HC,
     1 14X,1HD,/6H X-DIR4E15.6/ 6H Y-DIR4E15.6/ 6H Z-DIR4E15.6/ )
 2010 FORMAT(1H1,I5 ,
     1  30X40H FIXED END FORCES IN LOCAL COORDINATES
     2//53H TYPE   NODE         FORCE X       FORCE Y       FORCE Z
     3                     35H MOMENT X     MOMENT Y     MOMENT Z             )
 2011 FORMAT(1H ,I3,6X,1HI,3X,6F12.3/1H ,9X,1HJ,3X,6F12.3/)
 2013 FORMAT(1HC/
     1  60H SECTION PROPERTIES OTHER THAN SHEAR AREAS MAY NOT BE SPECIF
     2  34HIED AS ZERO. EXECUTION TERMINATED.)
 3000 FORMAT (1CI5,2I6,I8)
 4000 FORMAT(1H1/
     .. 5HOBEAM 5X 5HNODES 5X 5H MATL 5H GEOM 5X 10HELEM LOADS 4X   10X
     . 12H  END CODES / 5H  NO      4X 1HI 4X 1HJ 4X 1HK 5H  NO  5H  NO
     .  4X 1HA 4X 1HB 4X 1HC 4X 1HD 9X 1HI 9X 1HJ)
 4001 FORMAT (1CI5,2I10)
 4002 FORMAT (19HOBEAM NO ,I5, 26H     K NODE ON BEAM X-AXIS      ,
     .  26H......EXECUTION TERMINATED )
 4003 FORMAT(36HOELEMENT CARD ERROR, ELEMENT NUMBER= I6)
 4004 FORMAT(1H ,31HNODAL POINT NUMBERS FOR ELEMENT,I5,36HARE IDENTICAL.
     1 EXECUTION TERMINATED.)
 4005 FORMAT(8HOELEMENT,I5,39H HAS ZERO LENGTH. EXECUTION TERMINATED.)
      END


      SUBROUTINE NEWBM(E,G,RO,COPROP,SFT,NUMFIX,NUMETP)
C
C     FORM NEW BEAM STIFFNESS
C
      DIMENSION E(1),G(1),RO(1),COPROP(NUMETP,1),SFT(NUMFIX,1)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .    SF(12,4)
      COMMON /NEWB/ LC(4),T(3,3),JK(6),MELTYP,DL,MATTYP
      DIMENSION R(12),S(12,12),C(12)
C
      DO 5 I=1,144
    5 S(I)=C.
      AX =COPROP(MELTYP,1)
      AY =COPROP(MELTYP,2)
      AZ =COPROP(MELTYP,3)
      AAX=COPROP(MELTYP,4)
      AAY=COPROP(MELTYP,5)
      AAZ=COPROP(MELTYP,6)
      SHFY=C.0
      SHFZ=0.0
      ZY=E(MATTYP)/(DL*DL)
      EIY=ZY*AAY
      EIZ=ZY*AAZ
      IF(AY.NE.C.C) SHFY=6.*EIZ/(G(MATTYP)*AY)
      IF(AZ.NE.0.0) SHFZ=6.*EIY/(G(MATTYP)*AZ)
      COMMY=EIY/(1.+2.*SHFZ)
      COMMZ=EIZ/(1.+2.*SHFY)
C
C     FIXED END FORCES IN LOCAL COORDS
C
      DO 73 N=1,4
      M=LC(N)
      IF (M.GT.0)  GO TO 71
      DO 70 I=1,12
   70 SF(I,N)=0.
      GO TO 73
```

```
   71 DC 72 I=1,12
   72 SF(I,N)=SFT(N,I)
   73 CONTINUL
C
C     FORM ELEMENT STIFFNESS IN LOCAL COORDINATES
C
      S(1,1)= E(MATTYP)* AX/UL
      S(4,4)= G(MATTYP)*AAX/UL
      S(2,2)= CCMMZ*12./DL
      S(3,3)= CCMMY*12./DL
      S(5,5)= CCMMY* 4.*DL*(1.+0.5*SHFZ)
      S(6,6)= CCMMZ* 4.*DL*(1.+0.5*SHFY)
      S(2,6)= CCMMZ* 6.
      S(3,5)=-CCMMY* 6.
      DO 102 I=1,6
      J=I+6
  102 S(J,J)=S(I,I)
      DO 104 I=1,4
      J=I+6
  104 S(I,J)= S(I,I)
      S(6,12)= S(6,6)*(1.-SHFY)/(2.+SHFY)
      S(5,11)= S(5,5)*(1.-SHFZ)/(2.+SHFZ)
      S(2,12)= S(2,6)
      S(6, 8)=-S(2,6)
      S(8,12)= S(2,6)
      S(3,11)= S(3,5)
      S(5, 9)= S(3,5)
      S(9,11)= S(3,5)
      DO 106 I=2,12
      K=I-1
      DO 106 J=1,K
  106 S(I,J)=S(J,I)
C
C     MODIFY ELEMENT STIFFNESS AND FORCE FOR ZERO END FORCE CONDITION
C
      IF ((JK(1)+JK(2)).EQ.0) GO TO 145
      DO 14C K=1,2
      KK=JK(K)
      KD=100000
      I1=6*(K-1)+1
      I2=I1+5
      DO 14C I=I1,I2
      IF (KK.LT.KD)  GO TO 140
      SII=S(I,I)
      DO 125 N=1,12
  125 R(N)=S(I,N)
      DO 130 M=1,12
      C(M)=S(M,I)/SII
      DO 130 N=1,12
  130 S(M,N)=S(M,N)-C(M)*R(N)
      DO 135 N=1,4
      SFI=SF(I,N)
      DO 135 M=1,12
  135 SF(M,N)=SF(M,N)-C(M)*SFI
  136 KK=KK-KD
  140 KD=KD/10
  145 CONTINUE
C
C     FORM LOCAL FORCES TO GLOBAL DISPLACEMENTS ARRAY
C
      DO 31 I=1,288
   31 SA(I)=0.
      DO 15C LA=1,10,3
      LB=LA+2
      DO 15C MA=1,10,3
      MB=MA-1
      DO 150 I=LA,LB
      DO 15C JM=1,3
      J=JM+MB
      XX=0.
      DO 151 K=1,3
  151 XX=XX+S(I,K+MB)*T(K,JM)
  150 SA(I,J)=XX
C
C     ELEMENT STIFFNESS AND FORCE IN GOLBAL COORDS
C
      DO 32 I=1,576
   32 ASA(I)=0.
      DO 160 LA=1,10,3
      LB=LA-1
      DO 16C MA=1,10,3
      MB=MA+2
      DO 16C IL=1,3
      I=IL+LB
      DO 16C J=MA,MB
      XX=0.
      DO 161 K=1,3
  161 XX=XX+T(K,IL)*SA(K+LB,J)
  160 ASA(I,J)=XX
C
      DO 165 LA=1,10,3
      LB=LA-1
      DO 165 IL=1,3
      I=IL+LB
      DO 165 N=1,4
      XX=0.
      DO 162 K=1,3
  162 XX=XX-T(K,IL)*SF(K+LB,N)
  165 FF(I,N)=XX
C
C     FORM MASS MATRIX
C
      XXM=PC(MATTYP)*AX*DL/2.
      DO 180 M=1,3
      XM(M)=XXM
      XM(M+3)=0.
      XM(M+9)=0.
  180 XM(M+6)=XXM
      RETURN
      END
```

```
      SUBROUTINE SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
C     PERFORMS SLAVE...MASTER DISPLACEMENT TRANSFORMATION
C
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1)
      COMMON /EM/ LM(24),ND,NS,S(24,24),R(24,4),XM(24),SA(12,24),T(12,4)
C
C     DETERMINE REQUIRED TRANSLATION DEGREES OF FREEDOM
C
      DO 54 NF=1,12,6
      NOD=NI
      IF (NF.EQ.7) NOD=NJ
      DO 30 K=1,3
      I=K+NF-1
      IF (LM(I).GE.0) GO TO 30
      M=-LM(I)
      LM(I)=ID(M,K)
      IF(K-2) 35,45,55
   35 D1=-(Y(NOD)-Y(M))
      D2=  Z(NOD)-Z(M)
      LM(ND+1)=ID(M,6)
      LM(ND+2)=ID(M,5)
      GO TO 50
   45 D1=-(Z(NOD)-Z(M))
      D2=  X(NOD)-X(M)
      LM(ND+1)=ID(M,4)
      LM(ND+2)=ID(M,6)
      GO TO 50
   55 D1=-(X(NOD)-X(M))
      D2=  Y(NOD)-Y(M)
      LM(ND+1)=ID(M,5)
      LM(ND+2)=ID(M,4)
   50 CONTINUE
C
C     TRANSFORMATION...ARRAYS INCREASE IN SIZE
C
      DO 60 II=1,NC
      S(ND+1,II)=S(I,II)*D1
      S(ND+2,II)=S(I,II)*D2
      XM(ND+1)=XM(I)*D1*D1
      XM(ND+2)=XM(I)*D2*D2
      S(II,ND+1)=S(II,I)*D1
      S(II,ND+2)=S(II,I)*D2
      DO 56 J=1,4
      R(ND+1,J)=R(I,J)*D1
      R(ND+2,J)=R(I,J)*D2
   56 CONTINUE
   60 CONTINUE
C
      DO 70 II=1,NS
      SA(II,ND+1)=SA(II,I)*D1
   70 SA(II,ND+2)=SA(II,I)*D2
C
      S(ND+1,ND+1)=S(I,I)*D1**2
      S(ND+2,ND+2)=S(I,I)*D2**2
      S(ND+1,ND+2)=S(I,I)*D1*D2
      S(ND+2,ND+1)=S(ND+1,ND+2)
      ND=ND+2
   30 CONTINUE
C
C     SET ROTATIONS
C
      DO 54 J=1,3
      K=NF+J+2
      IF(LM(K).GE.0)  GO TO 54
      M=-LM(K)
      LM(K)=ID(M,J+3)
   54 CONTINUE
C
      RETURN
      END
```

```
      SUBROUTINE CBEAM
C
      COMMON A(1)
      COMMON /ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON / JUNK / MM,L,K,NTAG,NDYN,SIG(12),EXRA(188)
C
      IF(NPAR(1).EQ.0) GO TO 500
      N6=N5+NPAR(5)
      N7=N6+NPAR(5)
      N8=N7+NPAR(5)
      N9=N8+12*NPAR(4)
      N10=N5+6*NPAR(3)
      N11=N10+NPAR(6)
      IF (N11.GT.MTOT) CALL ERROR (N11-MTOT)
C
      CALL CBEAM (NPAR(2),NPAR(3),NPAR(4),NPAR(5),NPAR(6),
     *               A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),
     *               A(N9),A(N10),NUMNP,MBAND)
      RETURN
C
  500 IF(NTAG.EQ.0) WRITE (6,2002)
      WRITE(6,3002) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
C
 2002 FORMAT(/36H0      CURVED BEAM FORCES AND MOMENTS//
     * 10H0BEAM LOAD 5X 5HAXIAL   2(7X,5HSHEAR),5X 7HTORSION
     *  2(5X,7HBENDING)/ 10H  NO.  NO.  8X  2HR1  10X  2HR2  10X
     *  2HR3 10X 2HM1 10X 2HM2 10X 2HM3)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
      END


      SUBROUTINE CBEAM (NCBEAM,NUMETP,NUMFIX,NUMMAT,NUMRAD,ID,X,Y,Z,
     *                  E,G,RO,SFT,COPROP,RAD,NUMNP,MBAND)
C
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     *    SF(12,4)
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1),E(1),G(1),SFT(NUMFIX,1)
     *  ,COPROP(NUMETP,1),RO(1),EMUL(3,4),RAD(1)
      COMMON/NEWCB/LC(4),TI(3,3),TJ(3,3),JK(6),MELTYP,DL,MATTYP,S(12,12)
     *           ,BETA,B,DP,D,RA,AREA
      DIMENSION ILC(4),STIF(722),LS(4)
      EQUIVALENCE (STIF,LM)
C
C     FORM 3-D CURVED BEAM STIFFNESS AND STRESS-DISP. ARRAYS
C
      WRITE (6,2005) NCBEAM,NUMETP,NUMFIX,NUMMAT,NUMRAD
      N=0
      DO 5 I=1,1058
    5 STIF(I)=0.
C
C     READ AND PRINT RADIUS OF CURVATURE DATA
C
      WRITE (6,2000)
      DO 8 I=1,NUMRAD
      READ (5,1000) N,RAD(N)
      IF (RAD(N).NE.0.0) GO TO 7
      WRITE (6,2014)
      CALL EXIT
    7 WRITE (6,2015) N,RAD(N)
    8 CONTINUE
C
C     READ AND PRINT MATERIAL PROPERTY DATA
C
      WRITE (6,2001)
      DO 10 I=1,NUMMAT
      READ  (5,1001) N,E(N),G(N),RO(N)
      WRITE (6,2002) N,E(N),G(N),RO(N)
   10 G(N)=0.5*E(N)/(1.+G(N))
C
C     READ AND PRINT CROSS SECTIONAL PROPERTIES
C
      WRITE (6,2003)
      DO 30 I=1,NUMETP
      READ  (5,1002) N,(COPROP(N,J),J=1,6)
      IF((COPROP(N,1).NE.0.0).AND.(COPROP(N,4).NE.0.0).AND.
     1   (COPROP(N,5).NE.0.0).AND.(COPROP(N,6).NE.0.0))   GO TO 20
      WRITE (6,2013)
      CALL EXIT
   20 WRITE (6,2004) N,(COPROP(N,J),J=1,6)
   30 CONTINUE
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ  (5,1006) ((EMUL(I,J),J=1,4),I=1,3)
      WRITE (6,2006) ((EMUL(I,J),J=1,4),I=1,3)
C
C     READ AND PRINT FIXED END FORCES IN LOCAL COORDINATES
C
      IF(NUMFIX .EQ. 0) GO TO 56
      WRITE (6,2010)
      DO 55 I=1,NUMFIX
      READ  (5,1005) N,(SFT(N,J),J=1,12)
   55 WRITE (6,2011) N,(SFT(N,J),J=1,12)
   56 CONTINUE
C
C     READ AND PRINT ELEMENT DATA...NEED BE SUPPLIED FOR EACH ELEMENT
C
      WRITE (6,4000)
      L=0
   60 READ (5,3000) INEL,INI,INJ,INK,IMAT,IMEL,ILC,INELKI,INELKJ,IRAD
      L=L+1
      ML=INEL-L
      IF (ML)  66,67,66
   66 WRITE (6,4003) INEL
      CALL EXIT
   67 NEL=INEL
      NI=INI
      NJ=INJ
      NK=INK
      MATTYP=IMAT
      MELTYP=IMEL
      DO 90 I=1,4
   90 LC(I)=ILC(I)
      NEKODI=INELKI
      NEKODJ=INELKJ
      NR=IRAD
      WRITE (6,4001) NEL,NI,NJ,NK,MATTYP,MELTYP,LC,NEKODI,NEKODJ,NR
C
   74 DX=X(NJ)-X(NI)
```

```
      DY=Y(NJ)-Y(NI)
      DZ=Z(NJ)-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF(DL) 75,75,76
   75 WRITE (6,4005) NEL
      CALL EXIT
C
C     FORM GLOBAL TO LOCAL COORDINATE TRANSFORMATION.
C
   76 RA=RAD(NR)
      ARGU=DL/(2.0*RA)
      BETA=2.0*ASIN(ARGU)
      B =SIN(BETA)
      DP=COS(BETA)
      D =1.0-DP
      BX=X(NI)-X(NK)
      BY=Y(NI)-Y(NK)
      BZ=Z(NI)-Z(NK)
      BB=BX*BX+BY*BY+BZ*BZ
      BB=SQRT(BB)
      IF (BB.GT.0.0) GO TO 40
      WRITE (6,4002) INEL
      STOP
C
   40 CX=DY*BZ-DZ*BY
      CY=DZ*BX-DX*BZ
      CZ=DX*BY-DY*BX
      CC=CX*CX+CY*CY+CZ*CZ
      CC=SQRT(CC)
C
      BX=BX/BB
      BY=BY/BB
      BZ=BZ/BB
      CX=CX/CC
      CY=CY/CC
      CZ=CZ/CC
      AX=BY*CZ-BZ*CY
      AY=BZ*CX-BX*CZ
      AZ=BX*CY-BY*CX
C
      TI(1,1)=AX
      TI(1,2)=AY
      TI(1,3)=AZ
      TI(2,1)=BX
      TI(2,2)=BY
      TI(2,3)=BZ
      TI(3,1)=CX
      TI(3,2)=CY
      TI(3,3)=CZ
C
      TJ(1,1)=AX*DP-BX*B
      TJ(1,2)=AY*DP-BY*B
      TJ(1,3)=AZ*DP-BZ*B
      TJ(2,1)=AX*B+BX*DP
      TJ(2,2)=AY*B+BY*DP
      TJ(2,3)=AZ*B+BZ*DP
      TJ(3,1)=CX
      TJ(3,2)=CY
      TJ(3,3)=CZ
C
C     CHECK IF NEW STIFFNESS NEEDED
C
      IF (NEL.EQ.1) GO TO 80
      IF (NR.NE.NR)  GO TO 80
      IF  (ABS(DS-DL).GT.DL/100.)  GO TO 80
      IF ((MT.NE.MATTYP).OR.(ME.NE.MELTYP)) GO TO 80
      IF ((JK(1).NE.NEKODI).OR.(JK(2).NE.NEKODJ)) GO TO 80
      DO 81 I=1,4
      IF (LS(I).NE.LC(I)) GO TO 80
   81 CONTINUE
      GO TO 100
C
   80 DS=DL
      MT=MATTYP
      ME=MELTYP
      MR=NR
      DO 78 I=1,4
   78 LS(I)=LC(I)
      JK(1)=NEKODI
      JK(2)=NEKODJ
C
C     FORM NEW CURVED BEAM STIFFNESS
C
      CALL NEWCBM (E,G,RO,COPROP,SFT,RAD,NUMFIX,NUMETP)
C
  100 CALL TRANSFM (PJ)
C
C     ADD GRAVITY LOADING....POINT LOADS ONLY COMPUTED
C
      DO 180 I=1,3
      DO 180 J=1,4
      RF(I,J)=RF(I,J)+EMUL(I,J)*XM(I)
  180 RF(I+6,J)=RF(I+6,J)+EMUL(I,J)*XM(I+6)
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 170 M=1,6
      LM(M)=ID(NI,M)
      LM(M+12)=0
      LM(M+18)=0
  170 LM(M+6)=ID(NJ,M)
C
      NS=12
      ND=12
C
C     TRANSFORM TO MASTER DEGREES OF FREEDOM
C
      CALL SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITET (MBAND,NDIF)
C
C     CHECK FOR LAST ELEMENT
C
      IF (NCREAM-NEL) 66,500,60
  500 RETURN
C
 1000 FORMAT(I5,F10.0)
 1001 FORMAT(I5,3F10.0)
 1002 FORMAT(I5,6F10.0)
 1005 FORMAT(I5,6F10.0/F15.0,5F10.0)
 1006 FORMAT (4F10.0)
 2000 FORMAT (////
```

```
     .         27H RADIUS        RADIUS OF      /
     .        27H   NO           CURVATURE    //)
2001 FORMAT(55H1MATERIAL   YOUNG S     POISSON S            MASS
     .       /55H              MODULUS       RATIO              DENSITY      /
     .       //)
2002 FORMAT(1H ,I5,3X,F12.0,F14.5,F14.5)
2003 FORMAT(1H1/
    0  26H BEAM SECTIONAL PROPERTIES//
    1   48H ELEMENT        AREA          AREA           AREA
    2  60H INERTIA      INERTIA      INERTIA
    3 / 48H  TYPE               X      .     Y              Z
    4                    30H     X              Y              Z               //)
2004 FORMAT(1H ,I5,2X,6F12.3)
2005 FORMAT(39H1........THREE DIM. CURVED BEAM ELEMENTS//
     .  36H  NUMBER OF CURVED BEAMS                =,I5/
     .  36H  NUMBER OF SECTIONAL PROPERTY SETS=,I5/
     .  36H  NUMBER OF FIXED END FORCE SETS    =,I5/
     .  36H  NUMBER OF MATERIALS               =,I5/
     .  36H  NUMBER OF RADII                   =,I5)
2006 FORMAT(///25H ELEMENT LOAD MULTIPLIERS / 10X,1HA,14X,1HB,14X,1HC,
     .14X,1HD,//6H X-DIR4E15.6/ 6H Y-DIR4E15.6/ 6H Z-DIR4E15.6/ )
2010 FORMAT(1H1,1H ,
    1  30X40H FIXED END FORCES IN LOCAL COORDINATES
    2//53H TYPE   NODE          FORCE X      FORCE Y      FORCE Z
    3                       35H MOMENT X    MOMENT Y    MOMENT Z          //)
2011 FORMAT(1H ,I3,6X,1HI,3X,6F12.3/1H ,9X,1HJ,3X,6F12.3/)
2013 FORMAT(1H0/
    1  60H SECTION PROPERTIES OTHER THAN SHEAR AREAS MAY NOT BE SPECIF
    2  34HIED AS ZERO. EXECUTION TERMINATED.)
2014 FORMAT(1H0/
    1  60H RADIUS OF CURVATURE MAY NOT BE SPECIFIED
    2  34H     AS ZERO. EXECUTION TERMINATED.)
2015 FORMAT (I5,F20.5)
3000 FORMAT (10I5,2I6,I8)
4000 FORMAT(1H1/
     . 5H0BEAM 5X 5HNODES 5X 5H MATL 5H GEOM 5X 10HELEM LOADS 4X   10X
     . 12H  END CODES ,10X,6HRADIUS/
     .                         5H  NO      4X 1HI 4X 1HJ 4X 1HK 5H  ND  5H  NO
     .  4X IHA 4X IHB 4X 1HC 4X 1HD 9X 1HI 9X 1HJ ,12X,2HNO//)
4001 FORMAT (10I5,2I10,I14)
4002 FORMAT (9H0BEAM NO ,I5, 26H      K NODE ON BEAM X-AXIS      ,
     .  26H......EXECUTION TERMINATED )
4003 FORMAT(36H0ELEMENT CARD ERROR, ELEMENT NUMBER= I6)
4004 FORMAT(1H ,31HNODAL POINT NUMBERS FOR ELEMENT,I5,36HARE IDENTICAL.
     . EXECUTION TERMINATED.)
4005 FORMAT(8H0ELEMENT,I5,39H HAS ZERO LENGTH. EXECUTION TERMINATED.)
      END


      SUBROUTINE NEWCBM (E,G,RO,COPROP,SFT,RAD,NUMFIX,NUMETP)
C
C     FORM NEW CURVED BEAM STIFFNESS
C
      DIMENSION E(1),G(1),RO(1),COPROP(NUMETP,1),SFT(NUMFIX,1),RAD(1)
      COMMON/FM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .   SF(12,4)
      COMMON/NEWCB/LC(4),TI(3,3),TJ(3,3),JK(6),MELTYP,DL,MATTYP,S(12,12)
     .             ,BETA,B,DP,D,RA,AX
      DIMENSION R(12),C(12)
C
      DO 5 I=1,144
    5 S(I)=0.
       AX=CCPROP(MELTYP,1)
       AY=CCPROP(MELTYP,2)
       AZ=COPROP(MELTYP,3)
      AAX=CCPROF(MELTYP,4)
      AAY=CCPROP(MELTYP,5)
      AAZ=CCPROP(MELTYP,6)
C
      RA2=RA*RA
      RA3=RA2*RA
      EIY=E(MATTYP)*AAY
      EIZ=E(MATTYP)*AAZ
      PHIZ=AAZ/(AX*RA2)
      PHIY=EIY/(G(MATTYP)*AAX)
C
      IF (AY.LE.0.) GO TO 20
      ZETAZ=EIZ/(AY*G(MATTYP)*RA2)
      GO TO 21
   20 ZETAZ=0.
   21 IF (AZ.LE.0.) GO TO 30
      ZETAY=EIY/(AZ*G(MATTYP)*RA2)
      GO TO 31
   30 ZETAY=0.
   31 CONTINUE
      COMMY=RA3/EIY
      COMMZ=RA3/EIZ
C
C     FIXED END FORCES IN LOCAL COORDINATES
C
      DO 73 N=1,4
      M=LC(N)
      IF (M.GT.0)  GO TO 71
      DO 70 I=1,12
   70 SF(I,N)=0.
      GO TO 73
   71 DO 72 I=1,12
   72 SF(I,N)=SFT(M,I)
   73 CONTINUE
C
C     FORM ELEMENT STIFFNESS IN LOCAL COORDINATES
C
      AA=BETA*0.5 C.5*B*DP
      CC=BETA*C.5+C.5*B*DP
      EE=0.5*B*B
C
      F11=CCMMZ*(BETA+CC-2.0*B+PHIZ*CC+ZETAZ*AA)
      F12=CCMMZ*(D-EE-PHIZ*EE+ZETAZ*EE)
      F16=(COMMZ/RA)*(B-BETA)
      F22=CCMMZ*(AA+PHIZ*AA+ZETAZ*CC)
      F26=(COMMZ/RA)*(-D)
C
      F33=CCMMY*(AA+PHIY*(BETA+CC-2.0*B)+ZETAY*BETA)
      F34=(COMMY/RA)*(AA-PHIY*(B-CC))
      F35=(COMMY/RA)*(EE+PHIY*(D EE))
      F44=(COMMY/RA2)*(AA+PHIY*CC)
      F45=(COMMY/RA2)*(EE-PHIY*EE)
      F55=(COMMY/RA2)*(CC+PHIY*AA)
      F66=(COMMZ/RA2)*BETA
C
      U=F11*F22*F66+2.0*F12*F16*F26-F22*F16*F16-F11*F26*F26-F66*F12*F12
      W=F33*F44*F55+2.0*F34*F45*F35-F44*F35*F35-F33*F45*F45-F55*F34*F34
```

```
C
      S(1,1)= (F22*F66-F26*F26)/U
      S(1,2)=-(F12*F66-F16*F26)/U
      S(1,6)= (F12*F26-F22*F16)/U
      S(2,2)= (F11*F66-F16*F16)/U
      S(2,6)=-(F11*F26-F12*F16)/U
      S(6,6)= (F11*F22-F12*F12)/U
C
      S(3,3)= (F44*F55-F45*F45)/W
      S(3,4)=-(F34*F55-F35*F45)/W
      S(3,5)= (F34*F45-F44*F35)/W
      S(4,4)= (F33*F55-F35*F35)/W
      S(4,5)=-(F33*F45-F35*F34)/W
      S(5,5)= (F33*F44-F34*F34)/W
C
      S(1, 7)=-S(1,1)*OP+S(1,2)*B
      S(1, 8)=-S(1,1)*B-S(1,2)*DP
      S(1,12)= S(1,1)*RA*D+S(1,2)*RA*B-S(1,6)
      S(2, 7)=-S(1,2)*OP+S(2,2)*B
      S(2, 8)=-S(1,2)*B-S(2,2)*OP
      S(2,12)= S(1,2)*RA*D+S(2,2)*RA*B-S(2,6)
      S(3, 9)= S(3,3)
      S(3,10)= S(3,3)*RA*D-S(3,4)*OP+S(3,5)*B
      S(3,11)=-S(3,3)*RA*B-S(3,4)*B-S(3,5)*DP
      S(4, 9)= S(3,4)
      S(4,10)= S(3,4)*RA*D-S(4,4)*OP+S(4,5)*B
      S(4,11)=-S(3,4)*RA*B-S(4,4)*B - S(4,5)*DP
      S(5, 9)= S(3,5)
      S(5,10)= S(3,5)*RA*D-S(4,5)*DP+S(5,5)*B
      S(5,11)=-S(3,5)*RA*B-S(4,5)*B-S(5,5)*DP
      S(6, 7)= S(1,6)*DP+S(2,6)*B
      S(6, 8)=-S(1,6)*B-S(2,6)*DP
      S(6,12)= S(1,6)*RA*D+S(2,6)*RA*B-S(6,6)
C
      S( 7, 7)= S(1,1)
      S( 7, 8)=-S(1,2)
      S( 7,12)= S(1,6)
      S( 8, 8)= S(2,2)
      S( 8,12)=-S(2,6)
      S( 9, 9)= S(3,3)
      S( 9,10)= S(3,4)
      S( 9,11)=-S(3,5)
      S(10,10)= S(4,4)
      S(10,11)=-S(4,5)
      S(11,11)= S(5,5)
      S(12,12)= S(6,6)
C
      DO 106 I=2,12
      K=I-1
      DO 106 J=1,K
  106 S(I,J)=S(J,I)
C
C     MODIFY ELEMENT STIFFNESS AND FORCES FOR ZERO END FORCE CONDITION
C
      IF ((JK(1)+JK(2)).EQ.0) GO TO 145
      DO 140 K=1,2
      KK=JK(K)
      KO=100000
      I1=6*(K-1)+1
      I2=I1+5
      DO 140 I=I1,I2
      IF (KK.LT.KO)  GO TO 140
      SII=S(I,I)
      DO 125 N=1,12
  125 R(N)=S(I,N)
      DO 130 M=1,12
      C(M)=S(M,I)/SII
      DO 130 N=1,12
  130 S(M,N)=S(M,N)-C(M)*R(N)
      DO 135 N=1,4
      SFI=SF(I,N)
      DO 135 M=1,12
  135 SF(M,N)=SF(M,N)-C(M)*SFI
  136 KK=KK-KO
  140 KO=KO/10
  145 CONTINUE
      RETURN
      END


      SUBROUTINE TRANSFR (RO)
C
      DIMENSION RO(1)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XP(24),SA(12,24),
     .    SF(12,4)
      COMMON/NEWCB/LC(4),TI(3,3),TJ(3,3),JK(6),MELTYP,DL,MATTYP,S(12,12)
     .            ,BETA,B,DP,D,RA,AX
C
C     FORM LOCAL FORCE TO GLOBAL DISPLACEMENT ARRAY
C
      DO 31 I=1,288
   31 SA(I)=0.
C
      DO 150 LA=1,IC,3
      LB=LA+2
      DO 150 MA=1,IC,3
      MB=MA-1
      DO 150 I=LA,LB
      DO 150 JM=1,3
      J=JM+MB
      XX=0.
      IF (J.GT.6) GO TO 152
      DO 151 K=1,3
  151 XX=XX+S(I,K+MB)*TI(K,JM)
      GO TO 150
  152 DO 153 K=1,3
  153 XX=XX+S(I,K+MB)*TJ(K,JM)
  150 SA(I,J)=XX
C
C     ELEMENT STIFFNESS AND FORCES IN GLOBAL COORDS
C
      DO 32 I=1,576
   32 ASA(I)=0.
C
      DO 160 LA=1,IC,3
      LB=LA-1
      DO 160 MA=1,IC,3
      MB=MA+2
      DO 160 IL=1,3
      I=IL+LB
      DO 160 J=MA,MB
      XX=0.
```

```
      IF (I.GT.6) GC TO 162
      DO 161 K=1,3
  161 XX=XX+TI(K,IL)*SA(K+LB,J)
      GO TO 160
  162 DO 163 K=1,3
  163 XX=XX+TJ(K,IL)*SA(K+LB,J)
  160 ASA(I,J)=XX
C
      DO 170 LA=1,IC,3
      LB=LA-1
      DO 170 IL=1,3
      I=IL+LB
      DO 170 N=1,4
      XX=0.
      IF (I.GT.6) GC TO 172
      DO 171 K=1,3
  171 XX=XX-TI(K,IL)*SF(K+LB,N)
      GC TO 170
  172 DO 173 K=1,3
  173 XX=XX-TJ(K,IL)*SF(K+LB,N)
  170 PF(I,N)=XX
C
C     FCRM MASS MATRIX
C
      XXM=RC(MATTYP)*AX*RA*BETA/2.0
      DO 180 M=1,3
      XM(M)=XXM
      XM(M+3)=0.
      XM(M+S)=C.
  180 XM(M+6)=XXM
      RETURN
      END
```



```
      SUBROUTINE BOUND
C
      COMMON/ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,'FG
      COMMON/JUNK/  MM,L,K,NTAG,KDYN,SIG(12),EXRA(188)
      COMMON A(1)
C
      IF (NPAR(1).EC.0) GO TO 500
      N6=N5+NPAR(3)*6
      IF (N6.GT.MTOT) CALL ERROR (N6-MTOT)
      CALL BIND (NPAR(2),NPAR(3),A(N1),A(N2),A(N3),A(N4),A(N5),
     .           NUMNP,MBAND)
      RETURN
C
  500 IF (NTAG.EC.C) WRITE (6,2002)
      WRITE(6,3C02) MM,L,(SIG(I),I=1,6 )
      NTAG=1
      RETURN
C
 2002 FORMAT (/33HO ....BOUNDARY FORCES AND MOMENTS  //
     .  10HO BU. LOAD 5X 5HAXIAL 2(7X 5HSHEAR) 5X 7HTORSION
     .  2(5X 7HBENCING) /10H  NO    NO  8X 2HR1 10X 2HR2 10X 2H-3 10X
     .  2HM1 10X 2HM2 10X 2HM3 )
 3002 FORMAT (I5,I4,1PE11.3,5E12.3)
C
      END


      SUBROUTINE BIND (NBOUND,NSTIF,ID,X,Y,Z,STIF,NUMNP,MBAND)
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),STIF(NSTIF,1),T(3,3),
     .          SPRING(6)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),P(24,4),XM(24),SA(12,24),
     .          SF(12,4)
      CCMMCN/JUNK/  DATA(17),S(12,12)
C
C     FORM ELEMENT STIFFNESS OF BOUNDARY SPRING ELEMENTS
C
      DO 5 I=1,1058
    5 LM(I)=0.
      ND=6
      NS=6
      NE=0
      WRITE(6,2COC) NBOUND,NSTIF
C
C     READ AND PRINT SPRING STIFNESS SETS
C
      READ (5,1C10) (N,(STIF(N,J),J=1,6),N=1,NSTIF)
      WRITE(6,2C01)
      WRITE(6,2C10) (N,(STIF(N,J),J=1,6),N=1,NSTIF)
C
C     READ AND PRINT ELEMENT DATA---DATA NEED TO BE SUPPLIED FOR EACH
C
      WRITE(6,2CC4)
   10 NE=NE+1
      READ (5,1C3C) INE,NI,NJ,NK,NS
      IF (INE.EC.NE) GO TO 20
      WRITE(6,3COC) INE
      STOP
   20 WRITE(6,2040) NE,NI,NJ,NK,NS
```

```
C
C     FORM LOCAL TC GLOBAL TRANSFORMATION
C
      DX=X(NJ)-X(NI)
      DY=Y(NJ)-Y(NI)
      DZ=Z(NJ)-Z(NI)
      DD=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF (DD.GT.0.) GO TO 30
   25 WRITE(6,3001) INE
      STOP
   30 AX=X(NK)-X(NI)
      AY=Y(NK)-Y(NI)
      AZ=Z(NK)-Z(NI)
      BX=DY*AZ-DZ*AY
      BY=DZ*AX-DX*AZ
      BZ=DX*AY-DY*AX
      BB=SQRT(BX*BX+BY*BY+BZ*BZ)
      IF (BB.LE.0.) GO TO 25
C
      T(3,1)=BX/BB
      T(3,2)=BY/BB
      T(3,3)=BZ/BB
      T(1,1)=DX/DD
      T(1,2)=DY/DD
      T(1,3)=DZ/DD
      T(2,1)=T(3,2)*T(1,3)-T(3,3)*T(1,2)
      T(2,2)=T(3,3)*T(1,1)-T(3,1)*T(1,3)
      T(2,3)=T(3,1)*T(1,2)-T(3,2)*T(1,1)
C
C     FORM BOUNDARY STIFFNESSES IN LOCAL COORDINATE SYSTEM
C
      DO 100 I=1,144
  100 S(I)=C.
      DO 110 I=1,6
      SPRING(I)=STIF(NS,I)
      S(I,I)=SPRING(I)
  110 CONTINUE
C
C     FORM LOCAL TC GLOBAL STRESS ARRAY
C
      DO 12C I=1,288
  120 SA(I)=0.
      DO 15C LA=1,6,3
      LB=LA+2
      DO 15C MA=1,6,3
      MB=MA-1
      DO 15C I=LA,LB
      DO 15C JM=1,3
      J=JM+MB
      XX=0.
      DO 151 K=1,3
      YY=S(I,K+MB)
      IF (YY.EQ.0.) GO TO 151
      XX=XX+YY*T(K,JM)
  151 CONTINUE
  150 SA(I,J)=XX
C
C     FORM ELEMENT STIFFNESSES IN GLOBAL COORDINATE SYSTEM
C
      DO 155 I=1,576
  155 ASA(I)=0.
      DO 16C LA=1,6,3
      LB=LA-1
      DO 160 MA=1,6,3
      MB=MA+2
      DO 16C IL=1,3
      I=IL+LB
      DO 16C J=MA,MB
      XX=0.
      DO 161 K=1,3
      YY=SA(K+LB,J)
      IF (YY.EQ.0.) GO TO 161
      XX=XX+T(K,IL)*YY
  161 CONTINUE
  160 ASA(I,J)=XX
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 17C I=1,6
      LM(I)=ID(NI,I)
      LM(I+6)=0
      LM(I+12)=0
  170 LM(I+18)=C
C
C     WRITE ELEMENT INFORMATION CN TAPE
C
      CALL WRITET (MBAND,NDIF)
C
  250 IF (NE.LT.NBCUND) GO TO 10
      RETURN
C
 1010 FORMAT (I5,6F10.0)
 1030 FORMAT (6I5)
 2000 FORMAT (24H1......BOUNDARY ELEMENTS  ///
     .   28H NUMBER CF ELEMENTS          =,I5//
     =   28H NUMBER CF STIFFNESS SETS   =,I5)
 2001 FORMAT (4H1SET 20X 31HSTIFFNESSES OF BOUNDARY SPRINGS /
     .   4H NO  4X 2HR1 10X 2HR2 10X 2HR3 10X 2HM1 10X 2HM2 10X 2HM3 /)
 2004 FORMAT (5H1ELE. 5X 5HNODES 5X 5H MATL      /
     .   5H  NO  4X 1HI 4X 2HJ 4X 1HK 5H  NO       /)
 2010 FORMAT (I4,6E12.3)
 2040 FCRMAT (5I5)
 3000 FCRMAT (32H1*****BOUNDARY ELEMENT DATA NO. ,I3,15H IN WRONG ORDER /
     =          21H .....XEQ TERMINATED  )
 3001 FORMAT (35H1*****J/K NODE FOR BOUNDARY ELEMENT NC. ,I3,
     .          21HSPECIFIED INCORRECTLY  /21H .....XEQ TERMINATED  )
C
      END
```

```
      SUBROUTINE EXPJB
C
      COMMON A(1)
      COMMON/ELPAR/ NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON/JUNK/ MM,L,K,NTAG,NUYN,SIG(12),EXPA(188)
C
      IF (NPAR(1).EQ.0) GO TO 500
      CALL XPJB (NPAR(2),A(N1),A(N2),A(N3),A(N4),NUMNP,MBAND)
C
      RETURN
C
  500 IF (NTAG.EQ.0) WRITE(6,2002)
      WRITE(6,3002) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
C
 2002 FORMAT (/29HC....JOINT FORCES AND MOMENTS  //
     .   10HC JT. LOAD,5X,5HAXIAL,2(7X,5HSHEAR),5X,7HTORSION,
     .   2(5X,7HBENDING)/10H   NO.  NO. ,8X,2HR1,10X,2HR2,10X,2HR3,
     .   10X,2HM1,10X,2HM2,10X,2HM3)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
C
      END


      SUBROUTINE XPJB (NUMEL,ID,X,Y,Z,NUMNP,MBAND)
C
      COMMON/EP/ LM(24),ND,NS,ASA(24,24),P(24,4),XM(24),SA(12,24),
     .                 SF(12,4)
      COMMON/JUNK/ MPM(5),SIG(12),S(12,12),T(3,3),KD(6),XXX(29)
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1)
C
C     FORM 3-D STIFFNESSES FOR EXPANSION JOINT ELEMENTS
C
      DO 5 I=1,1058
    5 LM(I)=0.
      ND=12
      NS=12
      NE=0
C
C     INPUT EXPANSION JOINT DATA
C
      WRITE(6,2000) NUMEL
      WRITE(6,2010)
   10 NE=NE+1
      READ (5,1030) INE,NI,NJ,NK,NL,KD,TRACE
      IF (INE.EQ.NE) GO TO 15
      WRITE (6,4002)
      CALL EXIT
   15 IF (TRACE.EQ.0.) TRACE=1.0E+10
      IF (NK.EQ.0) READ (5,1040) XNK,YNK,ZNK
      IF (NL.EQ.0) READ (5,1040) XNL,YNL,ZNL
C
      WRITE(6,2020) NE,NI,NJ,NK,NL,KD
      IF (NK.EQ.0) GO TO 30
      XNK=X(NK)
      YNK=Y(NK)
      ZNK=Z(NK)
      GO TO 35
   30 WRITE(6,2060) XNK,YNK,ZNK
   35 IF (NL.EQ.0) GO TO 40
      XNL=X(NL)
      YNL=Y(NL)
      ZNL=Z(NL)
      GO TO 50
   40 WRITE(6,2070) XNL,YNL,ZNL
C
C     FORM LOCAL TO GLOBAL TRANSFORMATION
C
   50 DX=XNL-X(NI)
      DY=YNL-Y(NI)
      DZ=ZNL-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF (DL) 55,55,60
   55 WRITE(6,4000) NE
      CALL EXIT
C
   60 AX=XNK-X(NI)
      AY=YNK-Y(NI)
      AZ=ZNK-Z(NI)
C
      BX=AY*DZ-AZ*DY
      BY=AZ*DX-AX*DZ
      BZ=AX*DY-AY*DX
      BB=SQRT(BX*BX+BY*BY+BZ*BZ)
      IF (BB) 65,65,70
   65 WRITE(6,4001) NE
      CALL EXIT
C
   70 T(3,1)=BX/BB
      T(3,2)=BY/BB
      T(3,3)=BZ/BB
C
      AA=SQRT(AX*AX+AY*AY+AZ*AZ)
      IF (AA) 65,65,80
   80 T(2,1)=-AX/AA
      T(2,2)=-AY/AA
      T(2,3)=-AZ/AA
C
      T(1,1)=T(2,2)*T(3,3)-T(2,3)*T(3,2)
      T(1,2)=T(2,3)*T(3,1)-T(2,1)*T(3,3)
      T(1,3)=T(2,1)*T(3,2)-T(2,2)*T(3,1)
C
C     FORM JOINT STIFFNESS IN LOCAL COORDINATE SYSTEM
C
      DO 100 I=1,144
  100 S(I)=0.
      DO 110 I=1,6
      IF (KD(I).EQ.0) GO TO 110
      S(I,I)=TRACE
      S(I,I+6)=-TRACE
      S(I+6,I)=-TRACE
      S(I+6,I+6)=TRACE
  110 CONTINUE
C
C     FORM LOCAL-GLOBAL STRESS ARRAY
C
      DO 120 I=1,288
  120 SA(I)=0.
      DO 150 LA=1,10,3
      LB=LA+2
```

```
      DO 150 MA=1,10,3
      MB=MA-1
      DO 150 I=LA,LB
      DO 150 JM=1,3
      J=JM+MB
      XX=0.
      DO 151 K=1,3
      YY=S(I,K+MB)
      IF (YY.EQ.0.) GO TO 151
      XX=XX+YY*T(K,JM)
  151 CONTINUE
  150 SA(I,J)=XX
C
C     FORM JOINT STIFFNESS IN GLOBAL COORDINATE SYSTEM
C
      DO 155 I=1,576
  155 ASA(I)=0.
      DO 160 LA=1,10,3
      LB=LA-1
      DO 160 MA=1,10,3
      MB=MA+2
      DO 160 IL=1,3
      I=IL+LB
      DO 160 J=MA,MB
      XX=0.
      DO 161 K=1,3
      YY=SA(K+LB,J)
      IF (YY.EQ.0.) GO TO 161
      XX=XX+T(K,IL)*YY
  161 CONTINUE
  160 ASA(I,J)=XX
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 170 I=1,6
      LM(I)=ID(NI,I)
      LM(I+6)=ID(NJ,I)
      LM(I+12)=0
  170 LM(I+18)=0
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITET (MBAND,NDIF)
C
      IF (NE.LT.NUMEL) GO TO 10
C
      RETURN
 1030 FORMAT (5I5,6I1,F19.0)
 1040 FORMAT (3F10.0)
C
 2000 FORMAT (31H1....EXPANSION JOINT ELEMENT...  ///
     .  29H TOTAL NUMBER OF ELEMENTS...=  ,I5)
 2010 FORMAT (//4H  NE,5H   NI,5H   NJ,9H     NK   ,9H     NL   ,3X,
     .  6HJ.CODE  /)
 2020 FORMAT (I4,2I5,I6,3X,I6,6X,6I1  )
 2060 FORMAT (14X,F9.3/14X,F9.3/14X,F9.3)
 2070 FORMAT (23X,F9.3/23X,F9.3/23X,F9.3)
 4000 FORMAT (10H0  ELEMENT,I5,39H HAS ZERO LENGTH. EXECUTION TERMINATED
     ..  )
 4001 FORMAT (10H0  ELEMENT,I5,40H HAS WRONG K NODE. EXECUTION TERMINATE
     .D.  )
 4002 FORMAT (40H0...E.J. ELEMENT DATA IN WRONG ORDER... )
C
      END
```

APPENDIX III

FORTRAN IV LISTING OF PROGRAM NEABS

```
      PROGRAM NEABS (INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT,TAPE1,TAPE2,
     .               TAPE3=2100,TAPE4,TAPE7,TAPE8,TAPE9,TAPE10,TAPE11,
     .               TAPE12,TAPE13,TAPE14,TAPE15,TAPE30)
C
C
C     PROGRAM FOR NONLINEAR EARTHQUAKE ANALYSIS OF BRIDGE STRUCTURES
C
C
      COMMON/ELPAR/NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .             NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ GGCQ(2043)
      COMMON/MISC/ LG,LL,NFNR,NFN,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ HED(12),JUK(193)
      DIMENSION T(10)
C
C     PROGRAM CAPACITY CONTROLLED BY THE FOLLOWING TWO STATEMENTS...
C
      COMMON A(20000)
      MTOT=20000
C
C     PROGRAM CONTROL DATA
C
    5 CALL SECOND (T(1))
      READ (5,100) HED,NUMNP,NELTYP,NUMEL,NFN,NGM,NAT,NT,NOT,MSTEP
      IF (NUMNP.EQ.0) STOP
      WRITE(6,200) HED,NUMNP,NELTYP,NUMEL,NFN,NGM,NAT,NT,NOT,MSTEP
      IF (NAT.EQ.0) NAT=1
C
C     SETUP STRUCTURAL SYSTEM INPUTS
C
      CALL SETUP
      CALL SECOND (T(2))
C
C     STATIC ANALYSIS
C
      CALL STATIC
      CALL SECOND (T(3))
C
C     SETUP DYNAMIC LOAD INPUTS
C
      CALL LOADS
      CALL SECOND (T(4))
C
C     STEP-BY-STEP DYNAMIC ANALYSIS
C
      IF (MSTEP.EQ.1) CALL STEPC
      IF (MSTEP.EQ.2) CALL STEPB
      CALL SECOND (T(5))
C
C     OUTPUT RESULTS OF STEP-BY-STEP DYNAMIC ANALYSIS
C
      CALL OUTPUT
      CALL SECOND (T(6))
C
C     OUTPUT CP EXECUTION TIMES
C
      TT=0.
      DO 90 I=1,5
      II=I+1
      T(I)=T(II)-T(I)
   90 TT=TT+T(I)
      T(6)=TT
      WRITE(6,205) (T(I),I=1,6)
C
      STOP
C
  100 FORMAT (12A6/9I5)
  200 FORMAT (1H1,12A6 ///
     .            30H  NUMBER OF NODAL POINTS     = I5//
     .            30H  NUMBER OF ELEMENT TYPES    = I5//
     .            30H  NUMBER OF TOTAL ELEMENTS   = I5//
     .            30H  NUMBER OF TIME FUNCTIONS   = I5//
     .            30H  GROUND MOTION INPUT CODE   = I5//
     .            30H  NUMBER OF DELAY TIMES      = I5//
     .            30H  NUMBER OF TIME STEPS (DT)  = I5//
     .            30H  DT MULTIPLIER FOR OUTPUT   = I5//
     .            30H  STEP-BY-STEP SOLUTION CODE = I5 )
  205 FORMAT (21H1OVERALL TIME LOG.... ///
     .            33H  STRUCTURAL SYSTEM SETUP ...... ,F8.2//
     .            33H  STATIC ANALYSIS ............. ,F8.2//
     .            33H  SET UP DYNAMIC LOAD INPUTS ... ,F8.2//
     .            33H  STEP-BY-STEP DYNAMIC ANALYSIS . ,F8.2//
     .            33H  OUTPUT DYNAMIC RESPONSES ...... ,F8.2///
     .            33H  TOTAL SOLUTION TIME .......... ,F8.2 )
C
      END


      SUBROUTINE ERROR (MODE,N,PM)
C
C     PRINT APPROPRIATE ERROR MESSAGE...
C
      GO TO (1,2,3,4,5,6,7,8,9) MODE
C
C     ERROR MODE 1
C
    1 WRITE(6,2001) N,PM
      GO TO 100
C
C     ERROR MODE 2
C
    2 WRITE(6,2002) N,PM
      GO TO 100
C
C     ERROR MODE 3
C
    3 WRITE(6,2003) N,PM
      GO TO 100
C
C     ERROR MODE 4
C
    4 WRITE(6,2004) N,PM
      GO TO 100
C     ERROR MODE 5
C
    5 WRITE(6,2005) N,PM
      GO TO 100
C
C     ERROR MODE 6
C
    6 WRITE(6,2006) N,PM
```

```
      GO TO 100
C
C     ERROR MODE 7
C
    7 WRITE(6,2007) N,PH
      GO TO 100
C
C     ERROR MODE 8
C
    8 WRITE(6,2008) N,PH
      GO TO 100
C
C     ERROR MODE 9
C
    9 WRITE(6,2009) N,PH
  100 STOP
C
 2001 FORMAT (1H1,44H*****DIMENSION IN BLANK COMMON A EXCEEDED BY,I6,
     .  20H    AT SUBROUTINE    ,A7/27H .....EXECUTION TERMINATED.)
 2002 FORMAT (1H1,42H*****WRONG INPUT FOR TOTAL NO. OF ELEMENTS ,I6,
     .  20H    AT SUBROUTINE    ,A7/27H .....EXECUTION TERMINATED.)
 2003 FORMAT (1H1,48H*****ELEMENT NONLINEAR PARAMETER NO. INPUT WRONG,
     .  10H   FOR NO. ,I6,17H     ELEMENT OF     ,A7/
     .  27H .....EXECUTION TERMINATED.)
 2004 FORMAT (1H1,50H*****ELEMENT DATA INPUT IN WRONG ORDER DETECTED AT,
     .  3HNO.,I6,17H     ELEMENT OF     ,A7/27H .....EXECUTION TERMINATED.)
 2005 FORMAT (1H1,16H*****ELEMENT NO. ,I6,9H    OF    ,A7,12HHAS 0 LENGTH
     .  27H .....EXECUTION TERMINATED.)
 2006 FORMAT (1H1,16H*****ELEMENT NO. ,I6,9H    OF    ,A7,17HHAS WRONG K
     .NODE. /27H .....EXECUTION TERMINATED.)
 2007 FORMAT (1H1,37H*****DATA INPUT NOT IN ORDER FOR NODE,I6,
     .  15H    DETECTED AT ,A7/27H .....EXECUTION TERMINATED.)
 2008 FORMAT (1H1,41H*****MULTIPLE GROUND MOTION INPUT AT NODE,I6/
     .  50H        NOT COMPATIBLE WITH THAT OF JOINT INPUT CODE ,
     .  13H  DETECTED AT,A7/27H .....EXECUTION TERMINATED.)
 2009 FORMAT (1H1,25H*****NONLINEAR ELEMENT NO ,I6,16H    IS SINGULAR.
     .  15H    DETECTED AT ,A7/27H .....EXECUTION TERMINATED. )
      END
```

```
      SUBROUTINE SETUP
C
      COMMON/ELPAR/NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ QQCQ(2043)
      COMMON/MISC/ LG,LL,NFNR,NFA,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ JUK(205)
      COMMON A(1)
C
C     INPUT JOINT DATA----JOINT ID CODE ARRAY STORED ON TAPE 8
C
      N1=1
      N2=N1+NUMNP*6
      N3=N2+NUMNP
      N4=N3+NUMNP
      N5=N4+NUMNP
      IF (N5.GT.MTOT) CALL ERROR (1,N5-MTOT,7HINPUTM  )
      CALL INPUTM (A(N1),A(N2),A(N3),A(N4),NUMNP,NEQ,NEQG,NGM)
C
C     INPUT ELEMENT DATA AND FORM ELEMENT STIFFNESSES--
C     STIFFNESS ON TAPE 2, STRESS ON TAPE 1, NONLINEAR DATA ON TAPE 3.
C
      N6=N5+NUMEL
      IF (N6.GT.MTOT) CALL ERROR (1,N6-MTOT,7HELSTF  )
      CALL ELSTF (A(N5),NUMEL)
C
C     CHECK AVAILABLE STORAGE AND CALCULATE SYSTEM EQUATION BLOCK SIZE
C
      LG=NEQG-NEQ
      CALL BLOCES (NEQB)
C
      NBLOCK=(NEQ-1)/NEQB+1
      IF (NEQB.GT.NEQ) NEQB=NEQ
      WRITE(6,2000) NEQ,MBAND,NEQB,NBLOCK
C
C     INPUT NODAL LOADS AND MASSES----ON TAPE 10
C
      N3=N2+NUMNP
      N4=N3+NUMNP*6
      N5=N4+NUMNP*6
      N6=N5+NEQB
      N7=N6+NEQE
      IF (N7.GT.MTOT) CALL ERROR (1,N7-MTOT,7HINLM    )
      CALL INLM (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),NUMNP,NEQB,NEQ)
C
C     ASSEMBLE TOTAL STIFFNESS,LOAD---ON TAPE 4,  MASS---ON TAPE 9
C
      NE2B=2*NEQB
      N2=N1+NE2B*MBAND
      N3=N2+NE2B
      N4=N3+NE2B
      N5=N4+NE2B*LG
      IF (N5.GT.MTOT) CALL ERROR (1,N5-MTOT,7HADDSTF  )
      CALL ADDSTF (A(N1),A(N2),A(N3),A(N4),NE2B,MBAND,LG,NUMEL,NEQ,
     .             NBLOCK)
      RETURN
C
 2000 FORMAT (/////28H SYSTEM SETUP INFORMATION...  ///
     .          30H TOTAL NUMBER OF EQUATIONS    =  I5//
     .          30H BAND WIDTH                   =  I5//
     .          30H NO OF EQUATIONS IN ONE BLOCK=  I5//
```

```
     .          30H TOTAL NUMBER OF BLOCKS       = I5  )
      END



      SUBROUTINE INPUTM (ID,X,Y,Z,NUMNP,NEQ,NEQG,NGM)
C
      DIMENSION ID(NUMNP,6),X(1),Y(1),Z(1),NID(6)
C
C     NODAL POINT INPUT AND GENERATION
C
      NO=6
      WRITE(6,200)
      WRITE(6,204)
      KO=1
      DO 10 I=1,6
   10 NID(I)=0
   11 READ (5,100) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N),KN
      WRITE(6,203) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N),KN
      DO 30 I=1,6
      IF (ID(N,I)) 21,22,23
   21 IF (ID(N,I).EQ.-2) GO TO 25
      NID(I)=-1
      ID(N,I)=1
      GO TO 30
   22 IF (NID(I).EQ.-1) ID(N,I)=1
      GO TO 30
   23 NID(I)=0
      GO TO 30
   25 ID(N,I)=-1
   30 CONTINUE
C
C     CHECK IF GENERATION NEEDED THEN GENERATE NEW NODES
C
      IF (KO.EQ.1) GO TO 32
      IF (KN) 33,33,34
   32 KO=0
   33 CONTINUE
      NUMINT=1
      GO TO 45
   34 NUMINT=(N-NI)/KN
      DX=(X(N)- X(NI))/NUMINT
      DY=(Y(N)-Y(NI))/NUMINT
      DZ=(Z(N)-Z(NI))/NUMINT
      DO 41 J=1,NUMINT
      NN=NI+J*KN
      X(NN)=X(NN-KN)+DX
      Y(NN)=Y(NN-KN)+DY
      Z(NN)=Z(NN-KN)+DZ
C
C     SET DOF CODES SAME AS FIRST JOINT IN THE SERIES
C
      DO 40 JJ=1,6
      IF (ID(NI,JJ)-1) 37,36,35
   35 ID(NN,JJ)=ID(NI,JJ)+J*KN
      GO TO 40
   36 ID(NN,JJ)=ID(NI,JJ)
      GO TO 40
   37 IF (ID(NI,JJ).NE.-2) GO TO 38
      ID(NN,JJ)=ID(NI,JJ)
      GO TO 40
   38 ID(NN,JJ)=0
   40 CONTINUE
   41 CONTINUE
   45 NI=N
C
C     CHECK FOR LAST NODAL POINT
C
      IF (NUMNP-NI) 48,48,11
   48 CONTINUE
      WRITE(6,202)
      WRITE(6,204)
      DO 50 N=1,NUMNP
   50 WRITE(6,203) N,(ID(N,I),I=1,6),X(N),Y(N),Z(N)
C
C     COUNT AND NUMBER UNKNOWNS ---SET SLAVE NODE NEGATIVE OF MASTER
C
      NEQ=0
      DO 60 I=1,NUMNP
      DO 60 J=1,NO
      IF (ID(I,J).LT.0) GO TO 60
      IF (ID(I,J)-1) 57,58,59
   57 NEQ=NEQ+1
      ID(I,J)=NEQ
      GO TO 60
   58 ID(I,J)=0
      GO TO 60
   59 ID(I,J)=-ID(I,J)
   60 CONTINUE
C
C     ADD GROUND  BOUNDARY DOF
C
      NEQG=NEQ
      IF (NGM.NE.2) GO TO 90
      DO 80 I=1,NUMNP
      DO 80 J=1,NO
      IF (ID(I,J).NE.-1) GO TO 80
      NEQG=NEQG+1
      ID(I,J)=NEQG
   80 CONTINUE
C
C     PRINT INTERNAL EQUATION NUMBER
C
   90 WRITE(6,205)
      WRITE(6,206)
      WRITE(6,207) (N,(ID(N,I),I=1,6),N=1,NUMNP)
      REWIND 8
      WRITE (8) ID
      RETURN
C
  100 FORMAT (7I5,3F10.0,I5)
  200 FORMAT (31H1NODAL POINT DATA AS INPUT.....  //)
  202 FORMAT (31H1COMPLETE NODAL POINT DATA.....  //)
  203 FORMAT (I5,6I5,3F13.3,I5)
  204 FORMAT (5HONODE 6X 24HBOUNDARY CONDITION CODES 13X
     .  23HNODAL POINT COORDINATES /
     .  7H NUMBER 2X 1HX 4X 1HY 4X 1HZ 3X 2HXX 3X 2HYY 3X 2HZZ 12X 1HX
     .  12X 1HY 12X 1HZ  /)
  205 FORMAT (48H1EQUATION NUMBER OF NODAL DEGREES OF FREEDOM....  //)
  206 FORMAT (5HONODE 10X 15HEQUATION NUMBER 12X
     .          5H NODE 10X 15HEQUATION NUMBER  /
     .  7H NUMBER 2X 1HX 4X 1HY 4X 1HZ 3X 2HXX 3X 2HYY 3X 2HZZ  6X
     .  7H NUMBER 2X 1HX 4X 1HY 4X 1HZ 3X 2HXX 3X 2HYY 3X 2HZZ  /)
```

```
  207 FORMAT (7I5,6X,7I5)
      END

      SUBROUTINE ELSTF (IND,NEL)
C
      DIMENSION IND(NEL)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
C
C     FORM ELEMENT STIFFNESSES AND STRESS ARRAY FOR EACH ELEMENT TYPE
C
      REWIND 1
      REWIND 2
      REWIND 3
      NUME=NUMEL
      NUMEL=0
      NUMNEL=0
      MBAND=0
      DO 200 M=1,NELTYP
      READ (5,1001) NPAR
      MTYPE=NPAR(1)
C
      GO TO ( 1,2,3,4,5 ) MTYPE
C
C     THREE DIMENSIONAL TRUSS ELEMENTS
C
    1 CALL TRUSS
      GO TO 100
C
C     THREE DIMENSIONAL STRAIGHT BEAM ELEMENTS
C
    2 CALL BEAM
      GO TO 100
C
C     THREE DIMENSIONAL CURVED  BEAM ELEMENTS
C
    3 CALL CBEAM
      GO TO 100
C
C     THREE DIMENSIONAL BOUNDARY SPRING ELEMENTS
C
    4 CALL BOUND
      GO TO 100
C
C     EXPANSION JOINT FICTITIOUS ELEMENTS
C
    5 CALL EXPJE
C
  100 NUMEL=NUMEL+NPAR(2)
  200 CONTINUE
C
C     COUNT NUMBER OF NONLINEAR ELEMENTS
C
      IF (NUME.NE.NUMEL) CALL ERROR (2,NUMEL,THELSTF  )
      DO 300 I=1,NUMEL
      IF (IND(I).GT.0) NUMNEL=NUMNEL+1
  300 CONTINUE
      WRITE(6,2000) NUMNEL
      WRITE(6,2001)
      WRITE(6,2002) (I,IND(I),I=1,NUMEL)
      WRITE (8) IND
      RETURN
C
 1001 FORMAT (14I5)
 2000 FORMAT (1H1,38HTOTAL NUMBER OF NONLINEAR ELEMENTS   =  ,I5  ///)
 2001 FORMAT (1H ,35HELEMENT TYPE OF L/NL INDICATOR.....  //
     .        1H ,5(14H  ELEMENT IND )/1H ,5(14H  NUMBER  TYPE)/)
 2002 FORMAT (1H ,5(I7,I6,1X))
      END

      SUBROUTINE WRITET (MBAND,NDIF)
C
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/EM/LM(24),ND,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
C
C     CALCULATE BAND WIDTH AND WRITE ELEMENT INFORMATION ON TAPES
C
      MIN=100000
      MAX=0
      DO 450 L=1,ND
      IF (LM(L).EQ. 0 ) GO TO 450
      IF (LM(L).GT.NEQ) GO TO 450
      IF (LM(L).GT.MAX) MAX=LM(L)
      IF (LM(L).LT.MIN) MIN=LM(L)
  450 CONTINUE
      NDIF=MAX-MIN+1
      IF (NDIF.GT.MBAND) MBAND=NDIF
C
      WRITE (1) NS,ND,LM,ST,TT
      WRITE (2) LM,ND,NS,S,P,XM
C
      RETURN
      END

      SUBROUTINE BLCEQ (NEQB)
C
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/MISC/ LG,LL,NFNR,NFM,NGM,NAT,NT,NCT,MSTEP
      COMMON/JUNK/ N(10),NB,NSB,NSAB,NN
C
C     CHECK STORAGE FOR SETUP AND STATIC
C
      NEQB=(MTOT-1-13*NUMNP)/2
      NA=(MTOT-1)/(MBAND+LG+2)/2
      IF (NEQB.GT.NA) NEQB=NA
C
      LL=LG+1
   10 NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL*(2+(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      NB=(MTOT-NSBB-1)/(MBAND+LL+1)
      IF (NEQB.LE.NB) GO TO 20
      NEQB=NB
```

```
      GO TO 10
C
   20 NB=(MTOT-6*NUMNP-6*LL-1)/LL
      IF (NEQB.GT.NE) NEQB=NB
C
C     CHECK STORAGE FOR DYNAMIC LOAD INPUTS
C
      NB=(MTOT-NFN*NAT-6*NUMNP-1)/(2*NFN)
      IF (NEQB.GT.NE) NEQB=NB
C
      IF (NGM-1) 60,30,40
C
   30 NBLOCK=(NEQ-1)/NEQB+1
      NB=(MTOT-NFN*NAT-6*NUMNP-1)/(2*NFN+NBLOCK+1)
      GO TO 50
   40 NB=(MTOT-NFN*NAT-6*NUMNP-3*LG-1)/(LG+1)
C
   50 IF (NEQB.GT.NE) NEQB=NB
C
   60 NB=(MTOT NFN*NAT-NAT-3*LG-1)/(NFN*NAT+2*NFN+LG)
      IF (NEQB.GT.NE) NEQB=NB
C
      M =2*NFN*NAT+NAT+NFN*NT+10*NFN*NAT+1
      IF (M.GT.MTOT) CALL ERROR (1,M-MTOT,7HLOADOH   )
      NB=(MTOT-10*NFN*NAT-1)/(10+NFN*NAT)
      IF (NEQB.GT.NE) NEQB=NB
      NBLOCK=(NEQ-1)/NEQB+1
      NB=(NEQ-1)/NBLOCK+1
      IF (NEQB.GT.NE) NEQB=NB
C
C     CHECK STORAGE FOR STEP-BY-STEP SOLUTION IN CORE
C
      NC=NEQB*NBLOCK
      NN=1+NEQ*E+NUMNEL*26
      N(1)=NC*(MBAND+LG)
      N(2)=NN+2*NEQ*MBAND+3*NEQ*LG+2*LG+NFN*NAT+NUMEL
      N(3)=NN+2*NEQ*MBAND+3*NEQ*LG+2*LG+NFN*NAT+NEQ
      M=0
      DO 100 I=1,3
      IF (M.LT.N(I)) M=N(I)
  100 CONTINUE
C
      IF (MSTEP-1) 150,200,300
C
  150 IF (M.GT.MTOT) GO TO 160
      WRITE(6,2000) M
      STOP
  160 WRITE (6,2001) M
      STOP
C
  200 IF (M.GT.MTOT) CALL ERROR (1,M-MTOT,7HSTEP    )
      WRITE(6,2000) M
      GO TO 500
C
  300 IF (M.GT.MTOT) GO TO 400
      WRITE(6,2000) M
      MSTEP=1
      GO TO 500
C
C     CHECK STORAGE FOR STEP-BY-STEP SOLUTION IN BLOCKS
C
  400 WRITE(6,2001) M
      NB=(MTOT-NN-2*NEQ)/MBAND
      IF (NEQB.GT.NE) NEQB=NB
  410 NSB=NEQB*(MBAND+1)
      NSBB=NEQB*(2+(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      NB=(MTOT-NN-NSBB)/(MBAND+2)
      IF (NEQB.LE.NE) GO TO 420
      NEQB=NB
      GO TO 410
C
  420 IF (LG) 450,450,430
  430 NB=(MTOT-NN)/(MBAND+LG)/2
      IF (NEQB.GT.NE) NEQB=NB
  435 NSB=NEQB*(MBAND+LG)
      NSBB=NEQB*LG*(2+(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      NB=(MTOT-NN-NSBB)/(MBAND+LG+1)
      IF (NEQB.LE.NE) GO TO 440
      NEQB=NB
      GO TO 435
C
  440 NB=(MTOT-NN-2*LG-NFN*NAT)/(2*LG)
      IF (NEQB.GT.NE) NEQB=NB
C
  450 MSTEP=2
      NBLOCK=(NEQ-1)/NEQB+1
      NB=(NEQ-1)/NBLOCK+1
      IF (NEQB.GT.NE) NEQB=NB
  500 RETURN
C
 2000 FORMAT (55H1STORAGE CHECK OK FOR STEP-BY-STEP SOLUTION IN CORE....
     .        //17H STORAGE NEEDED =  I8 )
 2001 FORMAT (40H1FOR STEP-BY-STEP SOLUTION IN CORE......
     .            28H MTOT SHOULD BE INCREASED TO   I8)
      END


      SUBROUTINE INLM (ID,IN,TL,TM,B,TMASS,NUMNP,NECB,NEQ)
C
      DIMENSION ID(NUMNP,6),IN(NUMNP),TL(NUMNP,6),TM(NUMNP,6),B(NECB),
     .          TMASS(NEQB)
      COMMON/JUNK/ AT,KSHF,NE,N,NN,II,JJ,R(6)
C
      NT=10
      REWIND 10
      KSHF=0
      DO 5 I=1,NUMNP
      IN(I)=0
      DO 5 J=1,6
      TL(I,J)=0.
    5 TM(I,J)=0.
      DO 10 I=1,NECB
      B(I)=0.
   10 TMASS(I)=0.
C
C     INPUT NODAL MASSES
C
      NE=0
  100 READ (5,1001) N,M
      NE=NE+1
```

```
      IF (N.EQ.0) GO TO 150
      IF (NE.EQ.1) WRITE(6,2003)
      WRITE(6,2001) N,R
      DO 140 J=1,6
  140 TM(N,J)=TM(N,J)+R(J)
      IN(N)=1
      GO TO 100
C
C     INPUT NODAL LOADS
C
  150 NE=0
  200 READ (5,1001) N,R
      NE=NE+1
      IF (N.EQ.0) GO TO 300
      IF (NE.EQ.1) WRITE(6,2002)
      WRITE(6,2001) N,R
      DO 240 J=1,6
  240 TL(N,J)=TL(N,J)+R(J)
      IN(N)=1
      GO TO 200
C
C     ASSEMBLE LOADS AND MASSES IN BLOCKS
C
  300 DO 900 NN=1,NUMNP
      IF (IN(NN).EQ.0) GO TO 900
      DO 800 J=1,6
      II=ID(NN,J)
      IF (II) 800,800,400
  400 IF (II.GT.NEQ) GO TO 800
  500 JJ=II-KSHF
      IF (JJ.LE.NEQB) GO TO 700
      WRITE (NT) B,TMASS
      KSHF=KSHF+NEQB
      DO 600 I=1,NEQB
      B(I)=0.
  600 TMASS(I)=0.
      GO TO 500
  700 B(JJ)=TL(NN,J)
      TMASS(JJ)=TM(NN,J)
  800 CONTINUE
  900 CONTINUE
C
      WRITE (NT) B,TMASS
      RETURN
C
 1001 FORMAT (I5,5X,6F10.0)
 2001 FORMAT (I5,5X,6F12.3)
 2002 FORMAT (23H1NODAL POINT LOADS..... // 10H NODE        27X
     .  14HAPPLIED LOADS               / 10H  NO.        8X 2HRX 10X
     .    2HRY 10X 2HRZ 10X 2HMX 10X 2HMY 10X 2HMZ )
 2003 FORMAT (23H1NODAL POINT MASSES.... // 10H NODE        27X
     .  19HCONCENTRATED MASSES         / 10H  NO.        8X 2HMX 10X
     .    2HMY 10X 2HMZ 10X 3HI/X 9X 3HI/Y 9X 3HI/Z )
      END


      SUBROUTINE ADDSTF (A,B,TMASS,C,NE2B,MBAND,LG,NUMEL,NEQ,NBLOCK)
C
      DIMENSION A(NE2B,MBAND),B(NE2B),TMASS(NE2B),C(NE2B,LG),STIF(722)
      COMMON/EM/ LM(24),ND,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
      COMMON/JUNK/ STR(4),K,MB,NEBB,X,NSHIFT,LMI,LMM,NUME,NUM7,II,JJ,MM
      EQUIVALENCE (STIF,LM)
C
C     FORM GLOBAL EQUILIBRIUM EQUATIONS IN BLOCKS---2 BLOCKS AT A TIME
C
      NEQB=NE2B/2
      K=NEQB+1
      X=NBLOCK
      MB=SQRT(X)
      MB=MB/2+1
      NEBB=MB*NE2B
      NSHIFT=0
      NUM7=0
      MM=1
      L=1
C
      REWIND 4
      REWIND 9
      REWIND 10
      DO 40 I=1,4
   40 STR(I)=0.
      WRITE(6,2000)
      READ (5,1001) STR
      WRITE(6,2001) L,(STR(I),I=1,4)
C
      DO 1000 M=1,NBLOCK,2
      DO 100 I=1,NE2B
      DO 80 J=1,MBAND
   80 A(I,J)=0.
      IF (LG.EQ.0) GO TO 100
      DO 90 J=1,LG
   90 C(I,J)=0.
  100 CONTINUE
      READ (10) (B(I),I=1,NEQB),(TMASS(I),I=1,NEQB)
      IF (M.EQ.NBLOCK) GO TO 200
      READ (10) (B(I),I=K,NE2B),(TMASS(I),I=K,NE2B)
  200 CONTINUE
C
      REWIND 7
      REWIND 2
      NA=7
      NUME=NUM7
      IF (MM.NE.1) GO TO 250
      NA=2
      NUME=NUMEL
      NUM7=0
  250 DO 700 N=1,NUME
      READ (NA) STIF
      DO 600 I=1,ND
      LMI=LM(I)
      IF (LMI.GT.NEQ) GO TO 600
      LMN=I-LMI
      II=LMI-NSHIFT
      IF ((II.LE.0).OR.(II.GT.NE2B)) GO TO 600
      TMASS(II)=TMASS(II)+XM(I)
      DO 300 J=1,4
  300 B(II)=B(II)+P(I,J)*STR(J)
      DO 500 J=1,ND
      JJ=LM(J)
      IF (JJ.LE.NEQ) GO TO 350
      L=JJ-NEQ
      C(II,L)=C(II,L)+S(I,J)
```

```
      GO TC 500
  350 JJ=JJ+LPN
      IF (JJ) 5CC,5C0,400
  400 A(II,JJ)=A(II,JJ)+S(I,J)
  500 CONTINUE
  600 CONTINUE
C
C     DETERMINE IF ELEMENT INFORMATION IS TO BE PLACED ON TEMPORARY TAPE
C
      IF (MM.GT.1) GO TO 700
      DO 65C I=1,ND
      LMI=LM(I)
      IF (LMI.GT.NEC) GO TU 650
      II=LMI-NSHIFT
      IF((II.GT.NE2B).AND.(II.LE.NEBA)) GO TO 660
  650 CONTINUE
      GC TC 700
  660 WRITE (7) STIF
      NUM7=NUM7+1
  700 CONTINUE
C
      IF (LG) 8CC,8C0,900
  800 WRITE (4) ((A(I,J),I=1,NEQB),J=1,MBAND),(B(I),I=1,NEQB)
      WRITE (9) (TMASS(I),I=1,NEQB)
      IF (M.EQ.NBLCCK) GO TO 1000
      WRITE (4) ((A(I,J),I=K,NE2B),J=1,MBAND),(B(I),I=K,NE2B)
      WRITE (9) (TMASS(I),I=K,NE2B)
      GO TC 950
C
  900 WRITE (4) ((A(I,J),I=1,NEQB),J=1,MBAND),((C(I,L),I=1,NEQB),L=1,LG)
     .          ,(B(I),I=1,NEQB)
      WRITE (9) (TMASS(I),I=1,NEQB)
      IF (M.EQ.NBLCCK) GO TO 1000
      WRITE (4) ((A(I,J),I=K,NE2B),J=1,MBAND),((C(I,L),I=K,NE2B),L=1,LG)
     .          ,(B(I),I=K,NE2B)
      WRITE (9) (TMASS(I),I=K,NE2B)
C
  950 IF (MM.EQ.MB) MM=0
      MM=MM+1
 1000 NSHIFT=NSHIFT+NE2B
C
      RETURN
C
 1001 FORMAT (4F10.C)
 2000 FORMAT (////49H ELEMENT LOAD MULTIPLIER FOR STATIC ANALYSIS.....//
     . 10H STRUCTURE  6X 24H ELEMENT LOAD MULTIPLIER /
     . 10H LOAD CASE 7X 1HA 9X 1HB 9X 1HC 9X 1HD //)
 2001 FORMAT (I6,5X,4F10.3)
      END
```

```
      SUBROUTINE STATIC
C
      COMMON/ELPAR/NPAR(14),NUMNP,NELTYP,NUMEL,NUMKEL,NEW,NEWG,MBAND,
     .               NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ GCCQ(2043)
      COMMON/MISC/ LG,LL,NFAR,MFR,NGM,MAT,NT,NOT,NSTEP
      COMMON/JUNK/ JUK(205)
      COMMON A(1)
C
C     COMPUTE STATIC RESPONSE---SOLVE FOR DISPLACEMENT UNKNOWNS
C
      LL=LG+1
      NSB=(MBAND+LL)*NEQB
      NSBB=NEQB*LL*(2+(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      N2=N1+NECB*MBAND
      N3=N2+NEQB*LL
      N4=N3+NSBB
      N5=N4+NECB
      IF (N5.GT.MTOT) CALL ERROR (1,N5-MTOT,7HUSOL     )
      CALL USOL (A(N1),A(N3),A(N4),NEQB,MBAND,LL,NBLCCK,NSB,4,10,7,2,2)
C
C     OUTPUT STATIC DISPLACEMENTS
C
      N2=N1+6*NUMNP
      N3=N2+6*LL
      N4=N3+NECB*LL
      CALL PRINTD (A(N1),A(N2),A(N3),NUMNP,NEQB,LL,NBLCCK,NEQ,2)
C
C     OUTPUT STATIC STRESSES OF EACH ELEMENT
C
      N2=N1+NECB*LL
      N3=N2+NEQ
      N4=N3+NUMEL
      CALL STRESS (A(N1),A(N2),A(N3),NEQB,LL,NEQ,NUMEL)
C
      RETURN
      END
```

```
      SUBROUTINE PRINTD (ID,D,B,NUMNP,NEQB,LL,NBLCCK,NEQ,NT)
C
      DIMENSION ID(NUMNP,6),D(6,LL),B(NEQB,LL)
      COMMON/JUNK/ STR(4),M,LG,III,JJJ,I,K,S,PS,NID,NQ,N
C
C     PRINT DISPLACEMENTS OF STATIC ANALYSIS
C
      REWIND NT
      REWIND 8
      READ (8) ID
      M=NEC
      NC=NECB*NBLCCK
      LG=LL-1
      IF (LG.EQ.0) WRITE(6,2003)
      IF (LG.GT.0) WRITE(6,2005)
      KKK=0
      N=NUMNP
      S =3H SD
      PS=3HPSD
```

```
C
      DO 500 KK=1,NUMNP
      III=6
      JJJ=6
      I=6
      DO 250 II=1,6
      DO 100 L=1,LL
  100 D(I,L)=0.
      IF (M.GT.NG) GO TO 150
      IF (M.EQ.0 ) GO TO 150
      READ (NT) B
      NG=NG-NEQB
  150 NID=ID(N,I)
      IF (NID.EQ. 0 ) JJJ=JJJ-1
      IF (NID.LT. 0 ) III=III-1
      IF (NID.GT.NEQ) GO TO 240
      IF (NID.LT. 1 ) GO TO 250
      K=M-NG
      M=M-1
      DO 200 L=1,LL
  200 D(I,L)=B(K,L)
      GO TO 250
  240 LF=NID-NEQ
      D(I,LF)=1.0
  250 I=I-1
C
      IF (III.EQ.0) GO TO 400
      IF (JJJ.EQ.0) GO TO 480
      WRITE(6,2004) N,S,(D(I,LL),I=1,6)
      KKK=1
  300 IF (LG.EQ.0) GO TO 500
      WRITE(6,2008)  (PS,(D(I,L),I=1,6),L=1,LG)
      KKK=1
      GO TO 500
  400 WRITE(6,2006) N
      GO TO 500
  450 WRITE(6,2007) N
      GO TO 500
  480 IF (KKK.EQ.0) GO TO 500
      WRITE(6,2009) N,S,(D(I,L),I=1,6)
      KKK=0
      GO TO 300
  500 N=N-1
C
      RETURN
C
 2003 FORMAT (31H1PRINT OF STATIC DISPLACEMENTS.....  //
     .  6X 25H SD---STATIC DISPLACEMENT //
     .  5H NODE 5H TYPE 11X 1HX 11X 1HY 11X 1HZ 9X 2HXX 9X 2HYY 9X 2HZZ/
     .  5H   NO 5H CODE /)
 2004 FORMAT (1H ,I4,2X,A3,1P3E12.3,3E11.2)
 2005 FORMAT (53H1PRINT OF STATIC DISPL. AND PSEUDO-STATIC COEFF..... //
     .  6X 25H SD --STATIC DISPLACEMENT /
     .  5X 32H PSD--PSEUDO-STATIC DISPLACEMENT //
     .  5H NODE 5H TYPE 11X 1HX 11X 1HY 11X 1HZ 9X 2HXX 9X 2HYY 9X 2HZZ/
     .  5H   NO 5H CODE /)
 2006 FORMAT (1H0,I4,8X,18H...SLAVE NODE .... )
 2007 FORMAT (1H0,I4,8X,26H...FIXED NODE ...(ALL 0. ) )
 2008 FORMAT (1H0,6X,A3,1P3E12.3,3E11.2 /(7X,A3,3E12.3,3E11.2) )
 2009 FORMAT (1H ,I4,2X,A3,1P3E12.3,3E11.2)
      END
      SUBROUTINE STRESS (B,D,IND,NEQB,LL,NEQ,NUMEL)
C
      DIMENSION B(NEQB,LL),D(NEQ),IND(NUMEL)
      COMMON/EM/ ND,NS,LM(24),S(24,24),P(24,4),XM(24),SA(12,24),TT(12,4)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,MUMEL,NUMMEL,MEQ,NEQG,MBAND,
     .             MEQD,NBLOCK,MTOT,NNN(6)
      COMMON/JUNK/ STR(4),MM,L,K,NTAG,NDYN,SIG(12),EXRA(184)
C
C     COMPUTE AND PRINT STATIC STRESSES OF EACH ELEMENT
C
      NT=2
      REWIND NT
      READ (8) IND
      NQ=NEQB*NBLOCK
      DO 200 NN=1,NBLOCK
      READ (NT) B
      N=NEQB
      IF (NN.EQ.1) N=NEQB-(NQ-NEQ)
      NQ=NQ-NEQB
      DO 200 J=1,N
      I=NQ+J
  200 D(I)=B(J,LL)
C
      REWIND 1
      L=1
      NTYPE=0
      MM=1
C
      DO 1000 MN=1,NUMEL
      MTYPE=IND(MN)
      IF (MTYPE.LT.0) MTYPE=-MTYPE
      IF (NTYPE.EQ.MTYPE) GO TO 250
      WRITE(6,2000)
      NPAR(1)=0
      NTAG=0
      NTYPE=MTYPE
      MM=1
  250 READ (1) NS,ND,LM,SA,TT
C
C     COMPUTE STRESS FROM DISPLACEMENT AND ADD LOCAL STRESS
C
      DO 300 N=1,NS
      SIG(N)=0.
      DO 280 I=1,4
  280 SIG(N)=SIG(N)+TT(N,I)*STR(I)
      DO 300 J=1,ND
      JJ=LM(J)
      IF (JJ.GT.NEQ) GO TO 300
      IF (JJ) 300,300,290
  290 SIG(N)=SIG(N)+SA(N,J)*D(JJ)
  300 CONTINUE
C
C     PRINT STRESSES FOR EACH ELEMENT
C
      GO TO (1,2,3,4,5) MTYPE
C
    1 CALL TRUSS
      GO TO 900
C
```

```
    2 CALL BEAM
      GO TO 900
C
    3 CALL CBEAM
      GO TO 900
C
    4 CALL BOUND
      GO TO 900
C
    5 CALL EXPJR
C
  900 MM=MM+1
      WRITE (8) NS,SIG
 1000 CONTINUE
      RETURN
C
 2000 FORMAT (27H1 STATIC STRESS OUTPUT.....   )
      END
```

```
      SUBROUTINE USOL (A,B,MAXB,NEQB,MB,LL,NBLOCK,NSB,NORG,NBKS,NT1,
     .                 NT2,NKST)
      DIMENSION A(NSB),B(NSB),MAXB(NEQB)
C
      NC=MB+LL
      NBR=(MB-1)/NEQB+1
      INC=NEQB-1
      NMB=NEQB*MB
      N2=NT2
      N1=NT1
      REWIND NORG
      REWIND NBKS
C
C     REDUCE EQUATIONS  BLOCK-BY-BLOCK
C
      DO 900 N=1,NBLOCK
      IF (N.GT.1.AND.NBR.EQ.1) GO TO 110
      IF (NBR.EQ.1) GO TO 105
      REWIND N1
      REWIND N2
  105 NI=N1
      IF(N.EQ.1) NI=NORG
      READ (NI) A
  110 DO 300 I=1,NEQB
      D=A(I)
      IF(D) 115,300,120
  115 M=NEQB*(N-1)+I
      WRITE(6,2000) M,D
C
  120 II=I
      DO 125 J=2,NC
      II=II+NEQB
  125 A(II)=A(II)/D
      DO 130 J=I,NMB,NEQB
      IF (A(J).NE.0.) MAXB(I)=J
  130 CONTINUE
C
      JL=I+1
      IF (JL.GT.NEQB) GO TO 300
      II=I
      DO 200 J=JL,NEQB
      II=II+NEQB
      IF(II.GT.NMB) GO TO 200
      C=A(II)
      IF (C.EQ.0.0) GO TO 200
      C=C*A(I)
C
      KK=J
      MAX=MAXB(I)
      DO 150 JJ=II,MAX,NEQB
      A(KK)=A(KK)-C*A(JJ)
  150 KK=KK+NEQB
C
      KK=J +NMB
      JJ=I +NMB
      DO 175 L=1,LL
      A(KK)=A(KK)-C*A(JJ)
      KK=KK+NEQB
  175 JJ=JJ+NEQB
  200 CONTINUE
  300 CONTINUE
```

```
      WRITE (NBKS) A,MAXB
C
C     SUBSTITUTE INTO REMAINING EQUATIONS
C
      DO 800 NN=1,NBR
      IF(N+NN.GT.NBLOCK) GO TO 900
      NI=N1
      IF(N.EQ.1) NI=NORG
      IF(NN.EQ.NBR) NI=NORG
      READ (NI) B
      IL=1+NN*NECB*NEQB
      DO 700 I=1,NECB
      II=IL
      DO 690 K=1,NECB
      IF (II.GT.NMB) GO TO 690
      C=A(II)
      IF (C.EQ.0.0) GO TO 690
      C=C*A(K)
      MAX=MAXB(K)
C
      KK=I
      DO 640 JJ=II,MAX,NEQB
      B(KK)=B(KK)-C*A(JJ)
  640 KK=KK+NEQB
      KK=I+NMB
      JJ=K+NMB
      DO 650 L=1,LL
      B(KK)=B(KK)-C*A(JJ)
      KK=KK+NEQB
  650 JJ=JJ+NEQB
  690 II=II-INC
  700 IL=IL+NECB
C
      IF(NBR.NE.1) GO TO 750
      DO 740 I=1,NMB
  740 A(I)=B(I)
      GO TO 800
  750 WRITE (N2) B
  800 CONTINUE
      M=N1
      N1=N2
  900 N2=M
C
C     BACKSUBSTITUTION - RESULTS ON TAPE NRST
C
      LS=LL*NECB
      NEB=NECB*(NBR+1)
      NUM=NBR*NEQB
      MAX=NEB*LL
      DO 905 I=1,MAX
  905 B(I)=0.
      REWIND NRST
C
      DO 1000 N=1,NBLOCK
      BACKSPACE NBKS
      READ (NBKS) A,MAXB
      BACKSPACE NBKS
      DO 910 L=1,LL
      K=L*NEB
      DO 910 J=1,NLB
      I=K-NEQB
      B(K)=B(I)
  910 K=K-1
C
      I=NMB
      DO 920 L=1,LL
      K=(L-1)*NEB
      DO 920 J=1,NECB
      I=I+1
      K=K+1
  920 B(K)=A(I)
C
      DO 955 I=1,NECB
      J=NECB+1-I
      MAX=MAXB(J)
      IF (A(J).EQ.0.0) GO TO 955
      DO 950 L=1,LL
      KK=J+(L-1)*NEB
      JJ=KK+1
      IL=J+NEQB
      C=B(KK)
      DO 940 II=IL,MAX,NEQB
      C=C-A(II)*B(JJ)
  940 JJ=JJ+1
  950 B(KK)=C
  955 CONTINUE
C
      I=0
      DO 960 L=1,LL
      K=(L-1)*NEB
      DO 960 J=1,NECB
      K=K+1
      I=I+1
  960 A(I)=B(K)
C
      WRITE (NRST) (A(I),I=1,LS)
 1000 CONTINUE
      RETURN
C
 2000 FORMAT (32H0SET OF EQUATIONS MAY BE SINGULAR /
     * 26H DIAGONAL TERM OF EQUATION  I6, 8H  EQUALS  1PE12.4)
      END
```

```
      SUBROUTINE LOADS
C
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ AT(2043)
      COMMON/MISC/ LG,LL,NFNR,NFN,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ NARB,DT,ALFA,BETA,INTG,JUK(200)
      COMMON A(1)
C
C     READ AND PRINT LOAD INPUT CONTROLL DATA
C
      READ (5,1000) DT,ALFA,BETA,INTG
      WRITE(6,2000) DT,NT,NOT,NFN,NAT,NGM
      IF (NAT.EQ.0) NAT=1
      WRITE(6,2001) INTG,ALFA,BETA
C
C     INPUT DYNAMIC LOADS AT NODAL POINTS
C
      N1=1
      N2=N1+NFN*NAT
      N3=N2+NUMNP*6
      N4=N3+NEQB*NFN
      N5=N4+NEQB*NFN
      IF (N5.GT.MTOT) CALL ERROR (1,N5-MTOT,7HINDYL  )
      CALL INDYL (A(N1),A(N2),A(N3),A(N4),NEQB,NFN,NAT,NUMNP,NEQ)
C
      IF (NGM-1) 400,200,300
C
C     INPUT RIGID GROUND MOTION IN 3 DIRECTIONS
C
  200 N6=N5+NEQB
      N7=N6+NEQB*NBLOCK
      IF (N7.GT.MTOT) CALL ERROR (1,N7-MTOT,7HINRGM  )
      CALL INRGM (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),NFN,NUMNP,
     .            NEQB,NEQ,NBLOCK)
      GO TO 400
C
C     INPUT MULTIPLE GROUND MOTION PRESCRIBED AT EACH SUPPORTING POINTS
C
  300 N4=N3+NEQB*LG
      N5=N4+LG
      N6=N5+LG
      N7=N6+LG
      IF (N7.GT.MTOT) CALL ERROR (1,N7-MTOT,7HINMGM  )
      CALL INMGM (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),
     .            NFN,NAT,NUMNP,NEQB,LG,NEQ,NBLOCK   )
C
C     INPUT DELAY TIME OF LOAD FUNCTIONS
C
  400 N3=N2+NAT
      N4=N3+NAT
      N5=N4+NAT
      CALL INDLT (A(N1),A(N2),A(N3),A(N4),NFN,NAT,NFNR)
C
C     SET UP DYNAMIC LOAD MATRIX
C
      N4=N3+NEQB*NFNR
      N5=N4+NEQB*NFN
      N6=N5+NEQB*NFN
      N7=N6+LG
      N8=N7+LG
      N9=N8+LG
      N10=N9+NEQB*LG
      N11=N10+NEQB
      IF (N11.GT.MTOT) CALL ERROR (1,N11-MTOT,7HLOADM  )
      CALL LOADM (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9),
     .            A(N10),NFN,NAT,NFNR,NEQB,LG,NGM,NBLOCK)
C
C     INPUT TIME HISTORIES OF LOAD FUNCTION
C
      N4=N3+NFN*NT
      MAX=(MTOT-N4)/2
      IF (MAX.GE.NT) GO TO 440
      N5=2*(NT-MAX)
      CALL ERROR (1,N5,7HINHIS  )
  440 N5=N4+MAX
      CALL INHIS (A(N3),A(N4),A(N5),NFN,NT,MAX)
C
C     SET UP LOAD HISTORIES INCLUDING DELAY TIME EFFECT
C
      NTB=(MTOT-N4-NFNR)/NFNR
      NTBB=(MTOT-NEQB*NFNR)/(NEQB+NFNR)
      IF (NTB.GT.NTBB) NTB=NTBB
      NTBB=(MTOT-1)/(NEQB*NBLOCK)
      IF (NTB.GT.NTBB) NTB=NTBB
C
      IF (NTB.GT.NT) NTB=NT
      IF (NTB.GE.10) GO TO 450
      N5=N4+NFNR*10-MTOT
      N6=NEQB*NFNR+10*(NEQB+NFNR)-MTOT
      N7=NEQB*NBLOCK*10-MTOT
      IF (N5.LT.N6) N5=N6
      IF (N5.LT.N7) N5=N7
      CALL ERROR (1,N5,7HLOADH  )
C
  450 NB=(NT-1)/NTB+1
      N5=N4+NFNR*NTB
      N6=N5+NFNR
      CALL LOADH (A(N1),A(N2),A(N3),A(N4),A(N5),
     .            NFN,NAT,NFNR,NT,NTB,NB,NGM)
C
C     FORM DYNAMIC NODAL LOAD VECTOR
C
      N2=N1+NEQB*NTB
      N3=N2+NEQB*NFNR
      N4=N3+NFNR*NTB
      IF (N4.GT.MTOT) CALL ERROR (1,N4-MTOT,7HLOADV  )
      NQ=NEQB*NBLOCK
      N5=N1+NQ*NTB
      IF (N5.GT.MTOT) CALL ERROR (1,N5-MTOT,7HLOADV  )
      CALL LOADV (A(N1),A(N2),A(N3),A(N1),NEQB,NFNR,NTB,
     .            NBLOCK,NB,NT,NEQ,NQ)
C
      RETURN
 1000 FORMAT (3F10.0,I5)
 2000 FORMAT (37H1DYNAMIC LOAD INPUT CONTROL DATA..... ///
     .   32H  TIME INCREMENT DT   (SEC)      = F8.3//
     .   32H  TOTAL NUMBER OF TIME STEPS     = I5  //
     .   32H  OUTPUT INTERVAL                = I5  //
     .   32H  NUMBER OF TIME FUNCTIONS       = I5  //
     .   32H  NUMBER OF DELAY TIMES          = I5  //
     .   32H  GROUND MOTION INDICATOR        = I5    )
 2001 FORMAT (////48H STEP-BY-STEP DYNAMIC ANALYSIS CONTROL DATA.....///L-
```

```
     .  32H  INTEGRATION INDICATOR          =  I3  //
     .  32H  DAMPING FACTOR  ALPHA          =  1PE12.3//
     .  32H  DAMPING FACTOR  BETA           =  1PE12.3    )
C
      END



      SUBROUTINE INCYL (IOF,ID,FF,IFF,NEQB,NFN,NAT,NUMNP,NEQ)
C
      DIMENSION IOF(NFN,NAT),ID(NUMNP,6),FF(NEQB,NFN),IFF(NEQB,NFN)
      COMMON/JUNK/ NARB,DATA(4),NFAT,NNN,NP,IC,IFN,IAT,P,NN,II,NS,NE
C
C     INPUT ARBITRARY DYNAMIC LOADS AT NODAL POINTS
C
      MT=14
      REWIND MT
      REWIND 8
      READ (8) ID
      NFAT=NFN*NAT
      DO 5 I=1,NFAT
    5 IOF(I)=0
      NNN=NEQB*NFN
      DO 10 I=1,NNN
      FF(I)=0.
   10 IFF(I)=0
C
      NARB=1
      READ (5,1000) NP,IC,IFN,IAT,P
      IF (IAT.EQ.0) IAT=1
      IF (NP.GT.0) GO TO 50
      NARB=0
      RETURN
C
   50 WRITE(6,2000)
      WRITE(6,2002) NP,IC,IFN,IAT,P
      IF (IFN.GT.0) IOF(IFN,IAT)=1
C
      NS=1
      NE=NEQB
      DO 500 NN=1,NUMNP
      DO 500 II=1,6
      N=ID(NN,II)
      IF (N.GT.NEQ) GO TO 300
      IF (N.LE. 0 ) GO TO 300
      IF (N.GE.NS ) GO TO 100
      CALL ERROR (7,NP,7HINDYL   )
C
  100 IF (N.LE.NE ) GO TO 300
      WRITE (MT) FF,IFF
      DO 150 I=1,NNN
      FF(I)=0.
      IFF(I)=0
  150 CONTINUE
      NS=NS+NEQB
      NE=NE+NEQB
      GO TO 100
C
  300 IF ((NP.EQ.NN).AND.(IC.EQ.II)) GO TO 350
      GO TO 500
  350 IF (N.LE. 0 ) GO TO 400
      IF (N.GT.NEQ) GO TO 400
      M=N-NS+1
      IF (IFN.LE.0) GO TO 400
      FF(M,IFN)=P
      IFF(M,IFN)=IAT
  400 READ (5,1000) NP,IC,IFN,IAT,P
      IF (IAT.EQ.0) IAT=1
      IF (NP.LE.0) GO TO 300
      WRITE(6,2002) NP,IC,IFN,IAT,P
      IF (IFN.GT.0) IOF(IFN,IAT)=1
      GO TO 300
  500 CONTINUE
      WRITE (MT) FF,IFF
      RETURN
C
 1000 FORMAT (4I5,F10.0)
 2000 FORMAT (24H1DYNAMIC LOAD INPUT.....  //
     .  57H   NODE    DISPLACEMENT   FUNCTION    ARRIVAL TIME     FUNCTION /
     .  59H    NO.    COMPONENT      NUMBER          NUMBER       MULTIPLIER/)
 2002 FORMAT (I6,2I11,I14,F15.3)
C
      END



      SUBROUTINE INRGM (IOF,ID,FF,IFF,XM,MASS,NFN,NUMNP,NEQB,NEQ,NBLOCK)
C
      DIMENSION IOF(NFN,1),ID(NUMNP,1),FF(NEQB,NFN),IFF(NEQB,NFN),
     .          XM(NEQB),MASS(1)
      COMMON/JUNK/ NARB,DATA(4),JFN(3),PFN(3),JAT(3),JT,MT,NO,L,II,JJ
C
C     GENERATE PSEUDO-STATIC COEFFICIENTS FOR RIGID GROUND MOTION INPUT
C
      JT=9
      KT=10
      MT=14
      NT=15
      REWIND JT
      REWIND KT
      REWIND MT
      REWIND NT
      NC=NEQB*NBLOCK
      DO 5 I=1,NC
    5 MASS(I)=0
C
      L=1
      DO 50 N=1,NUMNP
      DO 40 I=1,6
      II=ID(N,I)
      IF (II.LE.0   ) GO TO 40
      IF (II.GT.NEQ) GO TO 40
      IF (I .GT.3   ) GO TO 30
      MASS(L)=I
   30 L=L+1
   40 CONTINUE
   50 CONTINUE
      WRITE (NT) (MASS(I),I=1,NEQ)
C
      READ (5,1000) JFN,JAT,PFN
      DO 100 I=1,3
      IF (JAT(I).EQ.0   ) JAT(I)=1
```

```
      JJ=JFN(I)
      II=JAT(I)
      IF (JJ.GT.0) IDF(JJ,II)=1
  100 CONTINUE
      WRITE(6,2000) JFN,JAT,PFN
C
      NNN=NEQB*NFN
      DO 500 N=1,NBLOCK
      NN=(N-1)*NEQB
      READ (JT) XM
      IF (NARB.EQ.0) GO TO 200
      READ (MT) FF,IFF
      GO TO 300
  200 DO 250 I=1,NNN
      FF(I)=0.
      IFF(I)=0
  250 CONTINUE
C
  300 DO 400 I*=1,NEQB
      I=NN+I*
      J=MASS(I)
      IF (J.LE.0) GO TO 400
      JJ=JFN(J)
      IF (JJ.LE.0) GO TO 400
      FF(II,JJ)=-XM(II)*PFN(J)
      IFF(II,JJ)=JAT(J)
  400 CONTINUE
      WRITE (MT) FF,IFF
  500 CONTINUE
      RETURN
C
 1000 FORMAT (6I5,3F10.0)
 2000 FORMAT (36H1GROUND MOTION INPUT KEY...... ///
     .  30X 9HDIRECTION /24X 1HX 7X 1HY 7X 1HZ  //
     .  20H  FUNCTION NUMBER... I5,2I8/20H  ARRIVAL TIME NO... I5,2I8 /
     .  20H  SCALE FACTOR...... 3F8.3 )
C
      END



      SUBROUTINE INPGM (IDF,ID,R,NFNG,PFNG,NATG,NFN,NAT,NUMNP,NEQB,
     .                  LG,NEQ,NBLOCK)
C
      DIMENSION IDF(NFN,1),ID(NUMNP,1),R(NEQB,LG),
     .          NFNG(LG),PFNG(LG),NATG(LG)
C
C     GENERATE PSEUDO-STATIC COEFFICIENTS FOR MULTI-GROUND-MOTION INPUT
C
      MT=2
      IT=13
      REWIND IT
      L=0
      DO 100 I=1,LG
      NFNG(I)=0
      PFNG(I)=0.
      NATG(I)=0
  100 CONTINUE
C
      WRITE(6,2000)
  150 L=L+1
      READ (5,1000) NP,IC,IFN,IAT,P
      IF (IAT.EQ.0) IAT=1
      IF (P.EQ.0.0) P=1.0
      IF ( NP.LE.0) GO TO 300
      WRITE(6,2001) NP,IC,IFN,IAT,P
      IF (IFN.GT.0) IDF(IFN,IAT)=1
C
      JJ=ID(NP,IC)
      IF (JJ.LE.NEQ) CALL ERROR (8,NP,7HINMGM   )
      JJ=JJ-NEQ
      NFNG(JJ)=IFN
      NATG(JJ)=IAT
      PFNG(JJ)=P
      GO TO 150
  300 IF (L.LE.LG) GO TO 350
      CALL ERROR (8,NP,7HINMGM   )
  350 WRITE (IT) NFNG,PFNG,NATG
C
      DO 500 N=1,NBLOCK
      BACKSPACE MT
      READ (MT) R
      BACKSPACE MT
      WRITE (IT) R
  500 CONTINUE
C
      RETURN
 1000 FORMAT (4I5,F10.0)
 2000 FORMAT (35H1MULTIPLE GROUND MOTION INPUT DATA.....  //
     .  54H  NODE   DISPLACEMENT  FUNCTION   ARRIVAL TIME  SCALE  /
     .  54H   NO.   COMPONENT     NUMBER        NUMBER     FACTOR /)
 2001 FORMAT (I6,2I11,I14,F12.3)
      END



      SUBROUTINE INCLT (IDF,LAT,KAT,AT,NFN,NAT,NFNP)
C
      DIMENSION IDF(NFN,NAT),LAT(NAT),KAT(NAT),AT(NAT)
      COMMON/JUNK/ NARB,DT,ALFA,BETA,INTG,JFN(3),PFN(3),JAT(3),IAT,NA,NF
C
C     INPUT DELAY TIMES
C
      READ (5,1000) (AT(I),I=1,NAT)
      WRITE(6,2000) (I,AT(I),I=1,NAT)
C
      DO 100 I=1,NAT
      AT(I)=AT(I)/DT
      IAT=AT(I)
      IF ((AT(I)-IAT).GE.0.5) IAT=IAT+1
      IAT=IAT+1
      LAT(I)=IAT
  100 CONTINUE
C
      DO 150 I=1,NAT
  150 KAT(I)=0
      IF (NAT.LT.2) GO TO 300
      NA=NAT-1
      DO 250 I=1,NA
      IF (KAT(I).GT.0) GO TO 250
      IAT=LAT(I)
      JJ=I+1
```

```
      DO 220 J=JJ,NAT
      IF (LAT(J).EQ.IAT) KAT(J)=I
  220 CONTINUE
  250 CONTINUE
C
C     COUNT TIME FUNCTION NUMBER INCLUDING DELAY TIME EFFECT
C
  300 NF=0
      DO 450 I=1,NFN
      DO 400 J=1,NAT
      IF (IDF(I,J).LE.0) GO TO 400
      JJ=KAT(J)
      IF (JJ.LE.0) GO TO 350
      IF (IDF(I,JJ).LE.0) GO TO 350
      IDF(I,J)=IDF(I,JJ)
      GO TO 400
  350 NF=NF+1
      IDF(I,J)=NF
  400 CONTINUE
  450 CONTINUE
      NFNR=NF
      RETURN
C
 1000 FORMAT (3F10.2)
 2000 FORMAT (/////15H DELAY TIMES...// 15X 7H DELAY  /8H  NUMBER 8X
     .  4HTIME /(I6,F14.2))
C
      END


      SUBROUTINE LOADM (IDF,LAT,FR,FF,IFF,NFNG,PFNG,NATG,R,XM,
     .                  NFN,NAT,NFNR,NEQB,LG,NGM,NBLOCK)
C
      DIMENSION IDF(NFN,NAT),LAT(NAT),FR(NEQB,NFNR),FF(NEQB,NFN),
     .          IFF(NEQB,NFN),NFNG(LG),PFNG(LG),NATG(LG),R(NEQB,LG),
     .          XM(NEQB)
      COMMON/JUNK/ NARB,DT,ALFA,BETA,INTG,JFN(3),PFN(3),JAT(3),NTREAD,
     .             IFN,IAT,NF
C
C     FORM LOAD MATRIX INCLUDING DELAY TIME EFFECT
C
      LT=9
      NT=10
      IT=11
      JT=12
      KT=13
      REWIND IT
      REWIND JT
      REWIND KT
      REWIND LT
      REWIND NT
      NTREAD=1
      IF ((NARB.EQ.0).AND.(NGM.NE.1)) NTREAD=0
      IF (NGM.NE.2) GO TO 20
      READ (KT) NFNG,PFNG,NATG
      DO 10 L=1,LG
      IFN=NFNG(L)
      IAT=NATG(L)
      IF (IFN.LE.0) GO TO 10
      IF (IAT.LE.0) GO TO 10
      NF=IDF(IFN,IAT)
      NFNG(L)=NF
   10 CONTINUE
      WRITE (IT) NFNG,PFNG,NATG
C
   20 IF (NGM.NE.1) GO TO 50
      DO 40 I=1,3
      IFN=JFN(I)
      IAT=JAT(I)
      IF (IFN.LE.0) GO TO 40
      IF (IAT.LE.0) GO TO 40
      NF=IDF(IFN,IAT)
      JFN(I)=NF
   40 CONTINUE
C
   50 NNR=NEQB*NFNR
      DO 800 N=1,NBLOCK
      DO 100 I=1,NNR
  100 FR(I)=0.
      IF (NTREAD.EQ.0) GO TO 400
      READ (NT) FF,IFF
      DO 300 I=1,NEQB
      DO 200 J=1,NFN
      IAT=IFF(I,J)
      IF (IAT.LE.0) GO TO 200
      NF=IDF(J,IAT)
      IF (NF.LT.1) GO TO 200
      FR(I,NF)=FR(I,NF)+FF(I,J)
  200 CONTINUE
  300 CONTINUE
  400 IF (NGM.NE.2) GO TO 700
      READ (LT) XM
      READ (KT) R
      WRITE (IT) R
      DO 600 L=1,LG
      NF=NFNG(L)
      IF (NF.LT.1) GO TO 600
      DO  500 I=1,NEQB
  500 FR(I,NF)=FR(I,NF)-XM(I)*R(I,L)*PFNG(L)
  600 CONTINUE
  700 WRITE (JT) FR
  800 CONTINUE
      RETURN
      END


      SUBROUTINE INFIS (PP,P,T,NFN,NT,MAXI)
C
      DIMENSION PP(NFN,NT),P(MAXI),T(MAXI)
      COMMON/JUNK/ NARB,DT,ALFA,BETA,INTG,JFN(3),PFN(3),NLP,SFTR,TINCR,
     .             HED(12),TL,TIME,OUT,DUP,SLOPE
C
C     INPUT LOAD TIME FUNCTIONS
C
      DO 500 I=1,NFN
      READ (5,1000) HED,NLP,SFTR,TINCR
      IF (SFTR.EQ.0.) SFTR=1.0
      WRITE(6,2000) I,HED,NLP,SFTR
      IF (NLP.LE.MAXI) GO TO 300
      L=2*(NLP-MAXI)
```

```
      CALL ERROR (1,L,THINHIS  )
  300 IF (TINCR.LE.0.) GO TO 350
      DO 310 L=1,NLP
      TL=L-1
  310 T(L)=TINCR*TL
      READ (5,1003) (P(L),L=1,NLP)
      GO TO 380
  350 READ (5,1001) (T(L),P(L),L=1,NLP)
  380 WRITE(6,2001) (T(L),P(L),L=1,NLP)
C
C     GENERATE EQUAL INTERVAL INCREMENTAL LOAD TIME FUNCTIONS
C
      TIME=DT
      TEMP=P(1)*SFTR
      L=1
      K=1
  410 L=L+1
      LL=L-1
  412 DDT=T(L)-T(LL)
      DDP=P(L)-P(LL)
      IF (DDT) 415,410,420
  415 WRITE(6,2003) T(L),P(L)
      L=L+1
      GO TO 412
  420 SLOPE=DDP/DDT
  423 IF (T(L)-TIME) 410,425,425
  425 PPP=P(LL)+(TIME-T(LL))*SLOPE
      PPP=PPP*SFTR
      PP(I,K)=PPP-TEMP
      TEMP=PPP
  430 TIME=TIME+DT
      K=K+1
      IF (NT-K) 500,423,423
  500 CONTINUE
      RETURN
C
 1000 FORMAT (12A6/I5,2F10.0)
 1001 FORMAT (12F6.0)
 1003 FORMAT (10F7.4,10X)
 2000 FORMAT (22H1TIME FUNCTION NO.....,I4,3X,12A6//
     .  6X 25HNUMBER OF LOAD POINTS    =  I6/
     .  6X 25HSCALE FACTOR             =  F10.3//)
 2001 FORMAT (5(19H   TIME    INPUT     )/(5(F7.3,F8.3,4X)))
 2003 FORMAT (19H    BAD LOAD DATA..  ,F7.3,F8.3,4X 12HDATA IGNORED )
      END



      SUBROUTINE LOADH (IDF,LAT,PP,PR,KR,NFN,NAT,NFNR,NT,NTB,NB,NGM)
C
      DIMENSION IDF(NFN,NAT),LAT(NAT),PP(NFN,NT),PR(NFNR,NTB),KR(NFNR)
C
C     FORM LOAD HISTORIES INCLUDING DELAY TIME EFFECT
C
      IT=2
      KT=10
      REWIND IT
      REWIND KT
      NS=1
      NE=NTB
      NK=NTB
      NNN=NFNR*NTB
C
      DO 800 N=1,NB
      DO 100 I=1,NNN
  100 PR(I)=0.
      DO 200 I=1,NFNR
  200 KR(I)=0
      DO 500 I=1,NFN
      DO 400 J=1,NAT
      NF=IDF(I,J)
      IF (NF.LE.0) GO TO 400
      IF (KR(NF).GT.0) GO TO 400
      JAT=LAT(J)
      NL=NS-JAT
      K=1
      IF (NL.LT.0) K=1-NL
      IF (K.GT.NK) GO TO 350
      DO 300 II=K,NK
      JJ=NL+II
  300 PR(NF,II)=PP(I,JJ)
  350 KR(NF)=1
  400 CONTINUE
  500 CONTINUE
      IF (NGM.EQ.0) GO TO 700
      DO 600 II=1,NK
  600 WRITE (KT) (PR(I,II),I=1,NFNR)
C
  700 WRITE (IT) PR
      NS=NS+NTB
      NE=NE+NTB
      IF (NE.GT.NT) NK=NT-NS+1
  800 CONTINUE
C
      RETURN
      END



      SUBROUTINE LOADV (FPR,FR,PR,DP,NEQB,NFNR,NTB,NBLOCK,NB,NT,NEQ,NG)
C
      DIMENSION FPR(NEQB,NTB),FR(NEQB,NFNR),PR(NFNR,NTB),DP(NG,NTB)
C
C     FORM TIME HISTORY OF NODAL INCREMENTAL LOAD VECTOR
C
      IT=2
      JT=7
      KT=12
      REWIND IT
      REWIND JT
      REWIND KT
C
      DO 500 NN=1,NB
      READ (IT) PR
      DO 400 N=1,NBLOCK
      READ (KT) FR
      DO 300 J=1,NTB
      DO 300 I=1,NEQB
      DO 300 K=1,NFNR
  300 FPR(I,J)=FR(I,K)*PR(K,J)
      WRITE (JT) FPR
  400 CONTINUE
```

```
      REWIND KT
  500 CONTINUE
C
      REWIND JT
      NL=NTB
      NK=NTB
      DO 800 NN=1,NB
      NS=1
      NE=NEQB
      DO 700 N=1,NBLOCK
      READ (JT) ((CP(I,J),I=NS,NE),J=1,NTB)
      NS=NS+NEQB
  700 NE=NE+NEQB
      IF (NL.GT.NT) NK=NT-(NN-1)*NTB
      DO 750 K=1,NK
  750 WRITE (KT) (DP(I,K),I=1,NEQ)
      NL=NL+NTB
  800 CONTINUE
C
      RETURN
      END
```

```
      SUBROUTINE STEPC
C
      COMMON/ELPAR/ CONT(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ QCCQ(2043)
      COMMON/MISC/ LG,LL,NFNR,NFN,NGM,MAT,NT,NOT,MSTEP
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),JUK(194)
      COMMON A(1)
C
C     STEP-BY-STEP NONLINEAR DYNAMIC ANALYSIS IN CORE
C
      N1=1
      N2=N1+NEQ
      N3=N2+NEQ
      N4=N3+NEQ
      N5=N4+NEQ
      N6=N5+NEQ
      N7=N6+NEQ
      N8=N7+NEQ
      N9=N8+NEQ*MBAND
      N10=N9+NEQ*LG
      N11=N10+NUMNEL
      N12=N11+NUMNEL
      N13=N12+NUMNEL*12
      N14=N13+NUMNEL*12
      N15=N14+NEQ
      N16=N15+NEQ*MBAND
      N17=N16+NEQ
      IF (NUMEL.GT.NEQ) N17=N16+NUMEL
      N18=N17+NEQ*LG
      N19=N18+NEQ*LG
      N20=N19+LG
      N21=N20+LG
      N22=N21+NFNR
      IF (N22.GT.MTOT) CALL ERROR (1,N22-MTOT,7HSTEPC   )
C
C     INITIALIZATION
C
      MB=MBAND+LG
      NC=NEQB*NBLOCK
      CALL START (A(N1),A(N1),A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),
     .            A(N8),A(N9),A(N10),A(N11),A(N12),A(N13),A(N15),A(N16),
     .            A(N18),A(N19),A(N20),MB,NC,MBAND,LG,NUMEL,NUMNEL,NFN)
      INDMAX=30
      IF (INDMAX.GT.NUMNEL) INDMAX=NUMNEL
      IF (NUMNEL.GT.0) WRITE(6,2000) (I,I=1,INDMAX)
C
C     SOLUTION BY STEP-BY-STEP INTEGRATION
C
      CALL INTGR (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9),
     .           A(N10),A(N11),A(N12),A(N13),A(N14),A(N15),A(N16),A(N17)
     ..          A(N18),A(N19),A(N20),A(N21),A(N14),A(N14),NEQ,MBAND,
     .           NUMNEL,LG,NFNR)
      RETURN
C
 2000 FORMAT (40H1STEP-BY-STEP ANALYSIS DATA MONITOR..... //
     .  12H STEP TAPE   20X 28HNONLINEAR ELEMENT INDICATORS/
     .  12H  NO  STIF        30I3  /)
C
      END
```

```
      SUBROUTINE START (B,D,X,X1,X2,XG,MASS,XM,OPU,AA,CC,NINU,NSS,STR,
     .            UPL,A,INO,R,NFNG,PFNG,MB,NQ,MBAND,LG,NUMEL,NUMNEL,NEQ)
C
      DIMENSION B(NQ,MB),D(NQ,LG),X(NEQ),X1(NEQ),X2(NEQ),XG(NEQ),
     .          MASS(NEQ),XM(NEQ),OPU(NEQ),NINO(NUMNEL),NSS(NUMNEL),
     .          STR(NUMNEL,12),UPL(NUMNEL,12),A(NEQ,MBAND),INO(NUMEL),
     .          R(NEQ,LG),NFNG(LG),PFNG(LG),AA(NEQ,MBAND),CC(NEQ,LG)
      COMMON/ELFAP/ CONT(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,NBAND,
     .             NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),NS,SIG(12),NB
      COMMON/COLLCMB/ CF(20,6)
      DATA CF /120*C.0/
C
C     GENERATE COMPUTATIONAL CONSTANTS
C
      DO 100 I=1,14
  100 CONT(I)=0.
C
      IF (INTG) 150,150,160
C
C     CONSTANT ACCELERATION SCHEME
C
  150 CONT(1)=4.C/DT/DT
      CONT(2)=2.C/DT
      CONT(3)=4.0/DT
      CONT(4)=2.C
      GO TC 200
C
C     LINEAR ACCELERATION SCHEME
C
  160 CONT(1)=6.C/DT/DT
      CONT(2)=3.0/DT
      CONT(3)=6.C/DT
      CONT(4)=3.C
      CONT(5)=DT/2.C
C
C     NORMALIZED CONSTANTS
C
  200 CONT(6)=1.0+BETA*CONT(2)
      CONT(7)=(CONT(1)+ALFA*CONT(2))/CONT(6)
      CONT(8)=1.C/CONT(6)
      CONT(9)=(CONT(3)+ALFA*CONT(4))/CONT(6)
      CONT(10)=BETA*CONT(4)/CONT(6)
      CONT(11)=(CONT(4)+ALFA*CONT(5))/CONT(6)
      CONT(12)=BETA*CONT(5)/CONT(6)
      CONT(13)=CONT(9)-CONT(7)*CONT(10)
      CONT(14)=CONT(11)-CONT(7)*CONT(12)
C
C     SET UP STIFFNESS MATRIX IN CORE
C
      REWINC 4
      NB=1
      NE=NEQB
      DO 300 N=1,NBLOCK
      READ (4) ((B(I,J),I=NB,NE),J=1,MB)
      NB=NB+NEQB
  300 NE=NE+NEQB
      REWINC 4
      WRITE (4) ((B(I,J),I=1,NEQ),J=1,MB)
C
      IF (LG.LE.0) GO TO 500
      REWIND 11
      READ (11)
      NB=1
      NE=NEQB
      DO 400 N=1,NBLOCK
      READ (11) ((D(I,L),I=NB,NE),L=1,LG)
      NB=NB+NEQB
  400 NE=NE+NEQB
      REWINC 11
      READ (11)
      WRITE (11) ((D(I,L),I=1,NEQ),L=1,LG)
C
C     INITIALIZE DISPLACEMENTS, VELOCITIES, ACCELERATIONS
C
  500 DO 510 I=1,NEQ
      X(I)=C.
      X1(I)=0.
      X2(I)=0.
      XG(I)=0.
  510 OPU(I)=0.
C
C     SET UP MASS MATRIX IN CORE
C
      NB=1
      NE=NEQB
      REWINC 9
      REWINC 15
      READ (15) (MASS(I),I=1,NEQ)
      DO 550 N=1,NBLOCK
      READ (9) (XM(I),I=NB,NE)
      NB=NB+NEQB
      NE=NE+NEQB
      IF (NE.GT.NEQ) NE=NEQ
  550 CONTINUE
C
C     INITIALIZE NONLINEAR ELEMENT STRESSES TO BE STATIC STRESSES
C
      READ (8) INC
      K=0
      DO 600 N=1,NUMEL
      READ (8) NS,SIG
      IF (INO(N).LE.0) GO TO 600
      K=K+1
      NINO(K)=C
      NSS(K)=NS
      DO 580 I=1,12
      STR(K,I)=SIG(I)
  580 UPL(K,I)=C.
  600 CONTINUE
C
      REWINC 4
      IF (LG) 7CC,7CO,750
  700 READ (4) ((AA(I,J),I=1,NEQ),J=1,MBAND)
      GO TC 800
  750 READ (4) ((AA(I,J),I=1,NEQ),J=1,MBAND),((CC(I,L),I=1,NEQ),L=1,LG)
  800 DO 900 I=1,NEQ
      DO 900 J=1,MBAND
  900 A(I,J)=AA(I,J)
C
      IF (LG.LE.0) RETURN
```

```
      REWIND 11
      READ (11) (NFNG(L),L=1,LG),(PFNG(L),L=1,LG)
      READ (11) ((R(I,L),I=1,NEQ),L=1,LG)
C
      RETURN
      END


      SUBROUTINE INTGR (X,X1,X2,XG,MASS,XM,DPU,B,D,NINO,NSS,STR,UPL,DX,A
     *          NBMAX,RT,R,NFNG,PFNG,PR,DP,XT,NEQ,MBAND,NUMNEL,LG,NFNR)
C
      DIMENSION X(NEQ),X1(NEQ),X2(NEQ),XG(NEQ),MASS(NEQ),XM(NEQ),
     .          DPU(NEQ),NINO(NUMNEL),NSS(NUMNEL),STR(NUMNEL,12),
     .          UPL(NUMNEL,12),DX(NEQ),A(NEQ,MBAND),NBMAX(NEQ),
     .          RT(NEQ,LG),R(NEQ,LG),NFNG(LG),PFNG(LG),PR(NFNR),
     .          DP(NEQ),XT(NEQ),B(NEQ,MBAND),D(NEQ,LG)
      COMMON/ELPAR/CONT(14),NNNN(17)
      COMMON/EM/ QQQQ(2043)
      COMMON/MISC/ LH,LL,MFNR,NFN,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),JUK(194)
C
C     STEP-BY-STEP INTEGRATION ALGORITHM
C
      REWIND 9
      REWIND 10
      REWIND 12
C
C     THE FOLLOWING TOLERANCE SHOULD BE RESET FOR LENGTH DIMENSION
C     OTHER THAN FT .........
      TOLOX=1.0E 1
C
      NSTIF=4
      NOT=1
      NCUT=NOT
C
C     FORM EFFECTIVE DYNAMIC STIFFNESS MATRIX AND LOAD VECTOR
C
  100 READ (12) (DP(I),I=1,NEQ)
      DO 120 I=1,NEQ
      DP(I)=DP(I)+DPU(I)
      DP(I)=CONT(8)*DP(I)+CONT(13)*XM(I)*X1(I)+CONT(14)*XM(I)*X2(I)
      A(I,1)=A(I,1)+CONT(7)*XM(I)
  120 CONTINUE
C
C     SOLVE FOR EFFECTIVE DYNAMIC DISPLACEMENT INCREMENTS
C
      CALL TRIA (A,NBMAX,NEQ,MBAND)
      CALL BACK (A,NBMAX,DP ,NEQ  )
C
C     COMPUTE NEW DISPLACEMENTS, VELOCITIES AND ACCELERATIONS
C
      DO 200 I=1,NEQ
      DD =DX(I)
      XX1=X1(I)
      XX2=X2(I)
      DXX=DD+CONT(10)*XX1+CONT(12)*XX2
      DX2=CONT(1)*DXX-CONT(3)*XX1-CONT(4)*XX2
      DX1=CONT(2)*DXX-CONT(4)*XX1-CONT(5)*XX2
      X (I)=X (I)+DXX
      X1(I)=X1(I)+DX1
      X2(I)=X2(I)+DX2
      DX(I)=DXX
  200 CONTINUE
C
      DO 300 I=1,NEQ
      IF (ABS(DX(I)).LT.TOLOX) GO TO 300
      NT=NOT-1
      WRITE(6,2000) NOT,(DX(I),I=1,NEQ)
      GO TO 1000
  300 CONTINUE
C
C     CHECK NONLINEARITY AND FORM NEW STIFFNESSES IF NEEDED
C
      CALL NELSTF (X,X1,DPU,B,D,NINO,NSS,STR,UPL,DX,A,RT,
     .             NEQ,MBAND,LG,NUMNEL,NOT,NOUT,NSTIF)
C
      IF (NGM-1) 900,800,400
C
  400 IF (NSTIF.EQ.2) GO TO 500
      GO TO 700
C
C     COMPUTE NEW PSEUDO-STATIC COEFF. FOR MULTI-GROUND MOTION INPUT
C
  500 CALL TRIA (A,NBMAX,NEQ,MBAND)
      DO 530 L=1,LG
      DO 510 I=1,NEQ
  510 DX(I)=RT(I,L)
      CALL BACK (A,NBMAX,DX,NEQ)
      DO 520 I=1,NEQ
  520 RT(I,L)=DX(I)
  530 CONTINUE
      REWIND NSTIF
      READ (NSTIF) ((A(I,J),I=1,NEQ),J=1,MBAND)
C
C     COMPUTE ABSOLUTE ACCELERATION
C
      READ (10) (PR(I),I=1,NFNR)
      DO 600 I=1,NEQ
      XX=0.
      YY=0.
      DO 580 L=1,LG
      NF=NFNG(L)
      IF (NF.LE.0) GO TO 580
      XX=XX+RT(I,L)*PFNG(L)*PR(NF)
      YY=YY-XM(I)*(RT(I,L)-R(I,L))*PFNG(L)*PR(NF)
  580 CONTINUE
      XG (I)=XG (I)+XX
      DPU(I)=DPU(I)+YY
  600 CONTINUE
      GO TO 850
C
  700 READ (10) (PR(I),I=1,NFNR)
      DO 720 I=1,NEQ
      XX=0.
      DO 710 L=1,LG
      NF=NFNG(L)
      IF (NF.LE.0) GO TO 710
      XX=XX+R(I,L)*PFNG(L)*PR(NF)
  710 CONTINUE
  720 XG(I)=XG(I)+XX
      GO TO 850
C
```

```
  800 READ (10) (PR(I),I=1,NFNR)
      DO 820 I=1,NEQ
      J=MASS(I)
      IF (J.LE.0) GO TO 820
      JJ=JFN(J)
      IF (JJ.LE.0) GO TO 820
      XG(I)=XG(I)+PFN(J)*PR(JJ)
  820 CONTINUE
C
C     STORE DISPLACEMENTS AND ABSOLUTE ACCELERATIONS ON TAPE
C
  850 DO 860 I=1,NEQ
  860 XT(I)=X2(I)+XC(I)
      IF (NCT.EQ.NCLT) WRITE (8) (X(I),I=1,NEQ),(XT(I),I=1,NEQ)
      GO TO 950
  900 IF (NCT.EQ.NCLT) WRITE (8) (X(I),I=1,NEQ),(X2(I),I=1,NEQ)
C
C     CHECK FOR LAST TIME STEP
C
  950 IF (NCT.EQ.NOUT) NOUT=NOUT+NOT
      NOT=NCT+1
      IF (NCT-NT) 100,100,1000
 1000 RETURN
C
 2000 FORMAT (50H1*****DISPLACEMENT INCREMENTS BLEW UP AT STEP NO  I6//
     .        29H      SKIP TO OUTPUT ROUTINE.  ///
     .         41H DISPLACEMENT INCREMENTS OF LAST STEP.... //(1P6E12.3))
C
      END


      SUBROUTINE TRIA (A,NBMAX,NEQ,MBAND)
C
      DIMENSION A(1),NBMAX(1)
C
C     TRIANGULARIZE BANDED MATRIX BY GAUSS ELIMINATION
C
      IF (NEQ.EQ.1) RETURN
      MM=NEQ*MBAND
      NE=NEQ-1
      DO 300 N=1,NE
C
C     DETERMINE VARIABLE BAND WIDTH
C
      NBMAX(N)=0
      DO 100 I=N,MM,NEQ
      IF (A(I).NE.0.0) NBMAX(N)=I
  100 CONTINUE
C
C     REDUCTION OF EQUATIONS WITHIN BAND
C
      IF (A(N).EQ.0.0) GO TO 300
      IL=N+NEQ
      IH=NBMAX(N)
      L=N
      DO 200 I=IL,IH,NEQ
      L=L+1
      IF (A(I).EQ.0.0) GO TO 200
      C=A(I)/A(N)
      J=L-I
      DO 150 K=I,IH,NEQ
  150 A(K+J)=A(K+J)-C*A(K)
      A(I)=C
  200 CONTINUE
  300 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE BACK (A,NBMAX,B,NEQ)
C
      DIMENSION A(1),NBMAX(1),B(1)
C
C     REDUCTION OF LOAD VECTOR AND BACKSUBSTITUTION
C
      IL=NEQ
      DO 400 N=1,NEQ
      C=B(N)
      IF (A(N).NE.0.0) B(N)=B(N)/A(N)
      IF (N.EQ.NEQ) GO TO 450
      IL=IL+1
      IH=NBMAX(N)
      K=N
      DO 350 I=IL,IH,NEQ
      K=K+1
  350 B(K)=B(K)-A(I)*C
  400 CONTINUE
C
  450 IL=2*NEQ
  500 IL=IL-1
      N=N-1
      IF (N.EQ.0) RETURN
      IH=NBMAX(N)
      K=N
      DO 600 I=IL,IH,NEQ
      K=K+1
  600 B(N)=B(N)-A(I)*B(K)
      GO TO 500
      END


      SUBROUTINE NELSTF (X,X1,DPU,B,D,NIND,NSS,STR,UPL,DX,A,C,
     .                   NEQ,MBAND,LG,NUMNEL,NOT,NOUT,NSTIF)
C
      DIMENSION X(1),X1(1),DX(1),NIND(1),NSS(NUMNEL),STR(NUMNEL,12),
     .          UPL(NUMNEL,12),DPU(1),C(NEQ,LG),A(NEQ,MBAND),STIF(1131),
     .          B(NEQ,MBAND),D(NEQ,LG)
      COMMON/EM/ MTYPE,LM(24),ND,NS,ASA(24,24),SA(12,24),S(12,12),
     .           ENPAR(24),TTT(72),ASSA(24,24),SSA(12,24),DF(24),
     .           F(12),UP(12)
      COMMON/NEM/ EX(24),EX1(24),DEX(24),DFF(12),DP(12),FF(12),P(12),
     .            SS(12,12),U(12)
      EQUIVALENCE (STIF,MTYPE)
C
C     COMPUTE NEW NONLINEAR ELEMENT STRESSES AND STIFFNESSES
C     ------ CHANGE GLOBAL EQUILIBRIUM EQUATIONS ACCORDINGLY
C
      DO 10 I=1,NEQ
   10 DPU(I)=0.
      DO 20 I=1,NEQ
      DO 20 J=1,MBAND
   20 A(I,J)=B(I,J)
      IF (LG.LE.0) GO TO 40
      DO 30 I=1,NEQ
      DO 30 L=1,LG
   30 C(I,L)=D(I,L)
   40 CONTINUE
      IF (NUMNEL.LE.0) GO TO 1000
C
      M=0
      REWIND 3
      DO 800 N=1,NUMNEL
      DO 50 I=1,72
   50 EX(I)=0.
      READ (3) STIF
      DO 100 I=1,ND
      II=LM(I)
      IF (II.LE.0.OR.II.GT.NEQ) GO TO 100
      EX (I)=X (II)
      EX1(I)=X1(II)
      DEX(I)=DX(II)
  100 CONTINUE
      DO 200 I=1,12
      F (I)=STR(N,I)
  200 UP(I)=UPL(N,I)
C
      IND=NIND(N)
      GO TO (1,2,3,4,5) MTYPE
C
C     NONLINEAR TRUSS ELEMENTS---SKIP
C
    1 GO TO 600
C
C     ELASTO-PLASTIC BEAM ELEMENTS
C
    2 CALL PBEAM (N,IND)
      GO TO 600
C
C     NONLINEAR CURVED BEAM ELEMENTS--SKIP
C
    3 GO TO 600
C
C     BILINEAR BOUNDARY SPRING ELEMENTS
C
    4 CALL PBOUND (N,IND)
      GO TO 600
C
C     NONLINEAR EXPANSION JOINT ELEMENTS
C
    5 M=M+1
      CALL NEXPJB (M,IND)
C
  600 NIND(N)=IND
C
C     ASSEMBLE NEW TOTAL STIFFNESSES AND UNBALANCED LOAD VECTOR
C
      IF (IND.EQ.0) GO TO 700
      DO 650 I=1,ND
      II=LM(I)
      IF (II.LE. 0 ) GO TO 650
      IF (II.GT.NEQ) GO TO 650
      DPU(II)=DPU(II)+DF(I)
      LMN=1-II
      DO 640 J=1,ND
      JJ=LM(J)
      IF (JJ.LE.NEQ) GO TO 625
      L=JJ-NEQ
      C(II,L)=C(II,L)+ASSA(I,J)-ASA(I,J)
      GO TO 640
  625 JJ=JJ+LMN
```

```
      IF (JJ) 640,640,635
  635 A(II,JJ)=A(II,JJ)+ASSA(I,J)-ASA(I,J)
  640 CONTINUE
  650 CONTINUE
C
C     UPDATE NONLINEAR ELEMENT STRESSES AND DEFORMATIONS
C
  700 NSS(N)=NS
      DO 750 I=1,12
      STR(N,I)=F (I)
  750 UPL(N,I)=UP(I)
  800 CONTINUE
C
      IF (NCT.NE.NCUT) GO TO 900
      WRITE (9) NSS,STR,UPL
      WRITE(6,2001) NOT,NSTIF,(NIND(I),I=1,NUMNEL)
  900 IF (LC.LE.0) GO TO 1000
      DO 910 N=1,NUMNEL
      IF (NIND(N).NE.0) GO TO 950
  910 CONTINUE
      NSTIF=4
      RETURN
C
  950 NSTIF=2
      REWIND 2
      WRITE (2) ((A(I,J),I=1,NEQ),J=1,MBAND)
 1000 RETURN
C
 2001 FORMAT (2I5,2X,(30I3))
C
      END
```

```
      SUBROUTINE STEPB
C
      COMMON/ELPAR/ CONT(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQB,MBAND,
     .               NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ QCCQ(2043)
      COMMON/MISC/ LG,LL,NFNR,NFN,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),JUK(194)
      COMMON A(1)
C
C     STEP-BY-STEP NONLINEAR DYNAMIC ANALYSIS IN BLOCKS
C
      LT=30
      NIT=1
      NDT=1
      NCUT=NOT
      NSTIF=4
C
      N1=1
      N2=N1+NEQ
      N3=N2+NEQ
      N4=N3+NEQ
      N5=N4+NEQ
      N6=N5+NEQ
      N7=N6+NEQ
      N8=N7+NEQ
      N9=N8+NEQ
C
      N10=NS +NUMNEL
      N11=N10+NUMNEL
      N12=N11+NUMNEL*12
      N13=N12+NUMNEL*12
C
      NC=NECB*NBLOCK
      N14=N13+NC
      N15=N14+NECB*MBAND
C
      LL=1
      NSB=NEQB*(MBAND+1)
      NSBB=NEQB+(2+(MBAND-1)/NEQB)
      IF (NSBB.LT.NSB) NSBB=NSB
      N21=N13+NSB
      N22=N21+NSBB
      N23=N22+NEQB
C
      N31=N13+NEQ
C
      NE2B=NEQB*2
      N41=N13+NE2B*MBAND
      N42=N41+NE2B*LG
C
      NGB=NEQB*(MBAND+LG)
      NGBB=NEQB*LG*(2+(MBAND-1)/NEQB)
      IF (NGBB.LT.NGB) NGBB=NGB
      N51=N13+NGB
      N52=N51+NGBB
      N53=N52+NEQB
C
      N61=N13+NEQB*LG
      N62=N61+NECB*LG
      N63=N62+LG
      N64=N63+LG
```

```
      N65=N64+NFNR
C
      IF (N15.GT.MTCT) CALL ERROR (1,N15 MTOT,7HSTEPB   )
      IF (N23.GT.MTCT) CALL ERROR (1,N23-MTOT,7HSTEPB   )
      IF (N31.GT.MTCT) CALL ERROR (1,N31-MTOT,7HNELSTB  )
      IF (N42.GT.MTCT) CALL ERROR (1,N42-MTOT,7HSTEPB   )
      IF (N53.GT.MTCT) CALL ERROR (1,N53-MTOT,7HSTEPB   )
      IF (N65.GT.MTCT) CALL ERROR (1,N65-MTOT,7HSTEPB   )
C
C     INITIALIZATICN
C
      CALL STARTB (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),
     .            A(N9),A(N10),A(N11),A(N12),A(N13),NEQ,NUMEL,NUMNEL)
C
      INDMAX=30
      IF (INDMAX.GT.NUMNEL) INDMAX=NUMNEL
      IF (NUMNEL.GT.0) WRITE(6,2000) (I,I=1,INDMAX)
C
C     STEP-BY-STEP SOLUTION IN BLOCKS
C
  100 CALL LOADP (A(N1),A(N2),A(N3),A(N7),A(N8),A(N13),
     .            A(N14),NEQ,NEQB,MBAND,NBLOCK,NQ,NSTIF)
C
      CALL USOL  (A(N13),A(N21),A(N22),NEQB,MBAND,LL,
     .            NBLOCK,NSB,7,13,14,15,15)
C
      CALL PACKB (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N8),
     .            A(N13),NEQ,NEQB,NBLOCK,NOT,NOUT,NIT,NGM,NFNR)
C
      IF (NIT.EC.C) GO TO 1000
      IF (NUMNEL.LE.0) GO TO 500
C
      CALL NELSTB (A(N1),A(N2),A(N8),A(N9),A(N10),A(N11),A(N12),
     .             A(N13),NEQ,NUMNEL,NOT,NOUT,NSTIF,LT )
C
      CALL ADDB  (A(N9),A(N13),A(N41),NEQ,NE2B,MBAND,
     .            LC,NEQB,NBLOCK,NUMNEL,NSTIF,LT )
C
  500 IF (NCM-1) 900,900,600
C
  600 IF (NSTIF.EQ.4) GO TO 800
      CALL USOL  (A(N13),A(N51),A(N52),NEQB,MBAND,LG,
     .            NBLOCK,NGB,2,13,14,15,15)
C
  800 CALL MGMB  (A(N1),A(N3),A(N4),A(N5),A(N7),A(N8),A(N13),A(N61),
     .            A(N62),A(N63),A(N64),NEQ,NEQB,LG,NFNR,NBLOCK,NOT,
     .            NCUT,NSTIF )
C
  900 IF (NCT.EC.NCUT) NOUT=NOUT+NOT
      NOT=NOT+1
      IF (NCT-NT) 100,100,1000
 1000 RETURN
C
 2000 FORMAT (40H1STEP-BY-STEP ANALYSIS DATA MONITOR..... //
     .  12H STEP TAPE   20X 28HNCNLINEAR ELEMENT INDICATORS /
     .  12H  NO  NSTIF       30I3  /)
C
      END
```

```
      SUBROUTINE STARTB (X,X1,X2,XG,XT,MASS,XM,OPU,NINO,NS,STR,UPL,
     .                   IND,NEQ,NUMEL,NUMNEL )
C
      DIMENSION X(NEQ),X1(NEQ),X2(NEQ),XG(NEQ),XT(NEQ),MASS(NEQ),XM(NEQ)
     .,      OPU(NEQ),NINO(NUMNEL),IND(NUMEL),NS(NUMNEL),STR(NUMNEL,12),
     .       UPL(NUMNEL,12)
      COMMON/ELPAR/ CONT(14),NUMNP,NELTYP,MUMEL,MUMAEL,MEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),NSS,SIG(12),
     .             NB,NE,JUK(179)
      COMMCN/CCLLCM8/ CF(20,6)
      DATA CF /120*C.0/
C
C     INITIALIZE DISPLACEMENTS, VELOCITIES, ACCELERATICNS AND UNBALANCED
C     FORCE VECTOR
C
      DO 10 I=1,NEG
      X (I)=0.
      X1(I)=0.
      X2(I)=0.
      XG(I)=0.
      XT(I)=0.
      OPU(I)=0.
   10 CCNTINUE
      REWIND 15
      READ (15) (MASS(I),I=1,NEQ)
C
C     INITIALIZE NCNLINEAR ELEMENT STRESS TO BE STATIC STRESS
C
      READ (8) IND
      K=0
      DC 400 N=1,NLMEL
      READ (8) NSS,SIG
      IF (IND(N).LE.0) GO TO 400
      K=K+1
      NIND(K)=0
      NS(K)=NSS
      DO 30C I=1,12
      STR(N,I)=SIG(I)
      UPL(N,I)=C.
  300 CCNTINUE
  400 CCNTINUE
C
C     GENERATE COMPUTATIONAL CONSTANTS
C
      DC 500 I=1,14
  500 CONT(I)=0.
C
      IF (INTG) 60C,600,700
C
C     CONSTANT ACCELERATION SCHEME
C
  600 CONT(1)=4.0/CT/DT
      CONT(2)=2.0/CT
      CONT(3)=4.0/CT
      CONT(4)=2.0
      GO TO 800
C
C     LINEAR ACCELERATION SCHEME
C
  700 CONT(1)=6.0/CT/DT
      CONT(2)=3.0/CT
```

```
      CONT(3)=6.0/CT
      CONT(4)=3.0
      CONT(5)=DT/2.C
C
C     NORMALIZED CONSTANTS
C
  800 CONT(6)=1.0+BETA*CONT(2)
      CONT(7)=(CONT(1)+ALFA*CONT(2))/CONT(6)
      CONT(8)=1.0/CONT(6)
      CONT(9)=(CONT(3)+ALFA*CONT(4))/CONT(6)
      CONT(10)=BETA*CONT(4)/CONT(6)
      CONT(11)=(CONT(4)+ALFA*CONT(5))/CONT(6)
      CONT(12)=BETA*CONT(5)/CONT(6)
      CONT(13)=CONT(9)-CONT(7)*CONT(10)
      CONT(14)=CONT(11)-CONT(7)*CONT(12)
C
C     SET UP MASS MATRIX IN CORE
C
      REWIND 9
      NB=1
      NE=NECB
      DO 950 N=1,NBLOCK
      READ (9) (XM(I),I=NB,NE)
      NB=NB+NECB
      NE=NE+NECB
      IF (NE.GT.NEQ) NE=NEQ
  950 CONTINUE
C
C     TAPE SETTING
C
      REWIND 9
      REWIND 10
      REWIND 12
C
      RETURN
      END


      SUBROUTINE LOADB (X,X1,X2,XM,DPU,DP,A,NEQ,NEQB,MBAND,NBLOCK,
     *                  NQ,NSTIF )
C
      DIMENSION X(1),X1(1),X2(1),XM(1),DPU(1),DP(NQ),A(NEQB,MBAND)
      COMMON/ELPAR/ CONT(14),NNNN(17)
C
C     FORM EFFECTIVE STIFFNESS MATRIX AND LOAD VECTOR
C
      REWIND 7
      REWIND NSTIF
      DO 10 I=1,NQ
   10 DP(I)=0.
      READ (12) (DP(I),I=1,NEQ)
      DO 20 I=1,NEC
   20 DP(I)=DP(I)+DPU(I)
C
      NS=1
      NE=NECB
      NK=NECB
      DO 800 N=1,NBLOCK
      READ (NSTIF) A
      DO 600 I=1,NK
      II=NS+I-1
      DP(II)=CONT(8)*DP(II)+CONT(13)*XM(II)*X1(II)
     *                     +CONT(14)*XM(II)*X2(II)
      A(I,1)=A(I,1)+CONT(7)*XM(II)
  600 CONTINUE
C
      WRITE (7) A,(DP(I),I=NS,NE)
      NS=NS+NEQB
      NE=NE+NEQB
      IF (NE.GT.NEQ)  NK=NEQ-NS+1
  800 CONTINUE
C
      RETURN
      END


      SUBROUTINE PACKB (X,X1,X2,XG,XT,MASS,DX,PR,NEC,NECB,NBLOCK,
     *                  NOT,NOUT,NIT,NGM,NFNR )
C
      DIMENSION X(1),X1(1),X2(1),XG(1),XT(1),MASS(1),DX(1),PR(1)
      COMMON/ELPAR/ CONT(14),NNNN(17)
      COMMON/JUNK/ NSTIF,DT,ALFA,BETA,INTG,JFN(3),PFN(3),NS,NE,
     *             DD,DX1,DX2,XX1,XX2,DXX
C
C     COMPUTE NEW DISPLACEMENTS, VELOCITIES, AND ACCELERATIONS
C
      NS=1
      NE=NECB
      DO 200 N=1,NBLOCK
      BACKSPACE 15
      READ (15) (DX(I),I=NS,NE)
      BACKSPACE 15
      NS=NS+NECB
      NE=NE+NECB
      IF (NE.GT.NEC) NE=NEQ
  200 CONTINUE
C
      DO 400 I=1,NEC
      DD =DX(I)
      XX1=X1(I)
      XX2=X2(I)
      DXX=DD*CONT(10)*XX1+CONT(12)*XX2
      DX2=CONT(1)*DXX-CONT(3)*XX1-CONT(4)*XX2
      DX1=CONT(2)*DXX-CONT(4)*XX1-CONT(5)*XX2
C
      X (I)=X (I)+DXX
      X1(I)=X1(I)+DX1
      X2(I)=X2(I)+DX2
      DX(I)=DXX
  400 CONTINUE
C
      DO 500 I=1,NEC
      IF (ABS(DX(I)).LT.1.0E+ 1) GO TO 500
      NIT=0
      NT=NOT-1
      WRITE(6,2000) NOT,(DX(I),I=1,NEQ)
      RETURN
  500 CONTINUE
C
      IF (NGM-1) 700,600,800
```

```
C
  600 READ (10) (PR(I),I=1,NFNR)
      DO 650 I=1,NEC
      J =MASS(I)
      IF (J.LE.0) GO TO 650
      JJ=JFN(J)
      IF (JJ.LE.0) GO TO 650
      XG(I)=XG(I)+PFN(J)*PR(J)
      XT(I)=X2(I)+XG(I)
  650 CONTINUE
C
      IF (NCT.EQ.NCLT) WRITE (8) (X(I),I=1,NEQ),(XT(I),I=1,NEQ)
      GO TO 800
C
  700 IF (NDT.EQ.NCLT) WRITE (8) (X(I),I=1,NEQ),(X2(I),I=1,NEQ)
  800 RETURN
C
 2000 FORMAT (50H1*****DISPLACEMENT INCREMENTS BLEW UP AT STEP NO  I6//
     .        29H *****SKIP TO OUTPUT ROUTINE.  ///
     .        41H DISPLACEMENT INCREMENTS OF LAST STEP.... //(1P6E12.3))
      END


      SUBROUTINE NELST8 (X,X1,DX,NIND,NSS,STR,UPL,DPU,NEQ,NUMNEL,
     .                   NDT,NOUT,NSTIF,LT )
C
      DIMENSION X(1),X1(1),DX(1),NIND(1),NSS(NUMNEL),STR(NUMNEL,12),
     .          UPL(NUMNEL,12),DPU(1),STIF(1131),KM(25)
      COMMON/EM/ MTYPE,LM(24),ND,NS,ASA(24,24),SA(12,24),S(12,12),
     .           ENPAR(24),TTT(72),ASSA(24,24),SSA(12,24),DF(24),
     .           F(12),UP(12)
      COMMON/NEM/ EX(24),EX1(24),DEX(24),DFF(12),DP(12),FF(12),P(12),
     .           SS(12,12),U(12)
      EQUIVALENCE (STIF,MTYPE)
C
C     COMPUTE ELEMENT STRESS INCREMENTS FOR NONLINEAR ELEMENTS
C
      REWIND 3
      REWIND LT
      DO 10 I=1,NEQ
   10 DPU(I)=0.
C
      DO 800 N=1,NUMNEL
      DO 20 I=1,72
   20 EX(I)=0.
      READ (3) STIF
      M=0
      DO 100 I=1,NC
      II=LM(I)
      IF (II.LE.0.OR.II.GT.NEQ) GO TO 100
      EX (I)=X (II)
      EX1(I)=X1(II)
      DEX(I)=DX(II)
  100 CONTINUE
      DO 200 I=1,12
      F (I)=STR(N,I)
      UP(I)=UPL(N,I)
  200 CONTINUE
C
      IND=NIND(N)
C
      GO TO (1,2,3,4,5) MTYPE
C
C     NONLINEAR TRUSS ELEMENTS--SKIP
C
    1 GO TO 600
C
C     ELASTC-PLASTIC BEAM ELEMENTS
C
    2 CALL FBEAM (N,IND)
      GO TO 600
C
C     NONLINEAR CURVED BEAM ELEMENTS--SKIP
C
    3 GO TO 600
C
C     BILINEAR BOUNDARY ELEMENTS
C
    4 CALL PBOUND (N,IND)
      GO TO 600
C
C     NONLINEAR EXPANSION JOINT ELEMENTS
C
    5 M=M+1
      CALL NEXPJB (M,IND)
C
  600 NIND(N)=IND
      IF (IND.NE.0) WRITE (LT) ND,LM,SSA,ASSA,ASA
C
      DO 650 I=1,NC
      II=LM(I)
      IF (II.LE. 0 ) GO TO 650
      IF (II.GT.NEQ) GO TO 650
      DPU(II)=DPU(II)+DF(I)
  650 CONTINUE
C
C     UPDATE NONLINEAR ELEMENT STRESSES AND DEFORMATIONS
C
      NSS(N)=NS
      DO 700 I=1,12
      STR(N,I)=F (I)
      UPL(N,I)=UP(I)
  700 CONTINUE
C
  800 CONTINUE
C
      WRITE(6,2001) NDT,NSTIF,(NIND(I),I=1,NUMNEL)
      DO 900 I=1,NEQ
  900 DX(I)=DPU(I)
      IF (NCT.NE.NCLT) RETURN
C
      WRITE (9) NSS,STR,UPL
      RETURN
C
 2001 FORMAT (2I5,2X,(30I3))
C
      END


      SUBROUTINE ACCB (NIND,A,C,NEQ,NE2B,MBAND,LG,NEGB,
```

```
                         NBLOCK,NUMNEL,NSTIF,LT)
C
      DIMENSION NINC(1),A(NE2B,MBAND),C(NE2B,LG),STIF(1465)
      COMMON/EM/ NO,LM(24),SA(12,24),S(24,24),SS(24,24),QQQQ(576)
      EQUIVALENCE (STIF,NO)
C
      DO 20 I=1,NUMNEL
      IF (NINO(I).NE.0) GO TO 50
   20 CONTINUE
      NSTIF=4
      RETURN
C
C     ASSEMBLE NEW GLOBAL EQUILIBRIUM EQUATIONS IN BLOCKS
C
   50 NSTIF=2
      K=NEQB+1
      NSHIFT=0
      REWIND 2
      REWIND 4
      DO 1000 M=1,NBLOCK,2
      IF (LG) 100,100,150
  100 READ (4) ((A(I,J),I=1,NEQB),J=1,MBAND)
      IF (M.EQ.NBLOCK) GO TO 200
      READ (4) ((A(I,J),I=K,NE2B),J=1,MBAND)
      GO TO 200
  150 READ (4) ((A(I,J),I=1,NEQB),J=1,MBAND),
     .         ((C(I,L),I=1,NEQB),L=1,LG    )
      IF (M.EQ.NBLOCK) GO TO 200
      READ (4) ((A(I,J),I=K,NE2B),J=1,MBAND),
     .         ((C(I,L),I=K,NE2B),L=1,LG    )
  200 CONTINUE
C
      REWIND L
      DO 700 N=1,NUMNEL
      IF (NINO(N).EQ.0) GO TO 700
C
      READ (LT) STIF
      DO 600 I=1,NC
      LMI=LM(I)
      IF (LMI.LE. 0 ) GO TO 600
      IF (LMI.GT.NEQ) GO TO 600
  300 LMN=1-LMI
      II=LMI-NSHIFT
      IF (II.LE.0.OR.II.GT.NE2B) GO TO 600
      DO 500 J=1,NC
      JJ=LM(J)
      IF (JJ.LE.NEQ) GO TO 350
      L=JJ-NEQ
      C(II,L)=C(II,L)+SS(I,J)-S(I,J)
      GO TO 500
  350 JJ=JJ+LMN
      IF (JJ) 500,500,400
  400 A(II,JJ)=A(II,JJ)+SS(I,J)-S(I,J)
  500 CONTINUE
  600 CONTINUE
  700 CONTINUE
C
      IF (LG) 800,800,900
  800 WRITE (2) ((A(I,J),I=1,NEQB),J=1,MBAND)
      IF (M.EQ.NBLOCK) GO TO 1000
      WRITE (2) ((A(I,J),I=K,NE2B),J=1,MBAND)
      GO TO 1000
  900 WRITE (2) ((A(I,J),I=1,NEQB),J=1,MBAND),
     .          ((C(I,L),I=1,NEQB),L=1,LG    )
      IF (M.EQ.NBLOCK) GO TO 1000
      WRITE (2) ((A(I,J),I=K,NE2B),J=1,MBAND),
     .          ((C(I,L),I=K,NE2B),L=1,LG    )
 1000 NSHIFT=NSHIFT+NE2B
C
      RETURN
      END


      SUBROUTINE MGMB (X,X2,XG,XT,XM,OPU,RT,R,NFNG,PFNG,PR,NEQ,NEQB,
     .                 LG,NFNR,NBLOCK,NDT,NOUT,NSTIF )
C
      DIMENSION XM(1),DPU(1),RT(NEQB,LG),R(NEQB,LG),NFNG(LG),PFNG(LG),
     .          PR(NFNR),X(1),X2(1),XG(1),XT(1)
C
C     MODIFY UNBALANCED FORCE VECTOR DUE TO CHANGE OF PSEUDO-STATIC COEF
C
      REWIND 11
      READ (11) NFNG,PFNG
      READ (10) PR
C
      NS=1
      NE=NEQB
      NK=NEQB
      DO 500 N=1,NBLOCK
      READ (11) R
      IF (NSTIF.EQ.2) GO TO 250
C
      DO 200 I=1,NK
      II=NS+I-1
      XX=0.
      DO 100 L=1,LG
      NF=NFNG(L)
      IF (NF.LE.0) GO TO 100
      XX=XX+R(I,L)*PFNG(L)*PR(NF)
  100 CONTINUE
  200 XG(I)=XG(I)+XX
      GO TO 450
C
  250 BACKSPACE 15
      READ (15) RT
      BACKSPACE 15
      DO 400 I=1,NK
      II=NS+I-1
      XX=0.
      YY=0.
      DO 300 L=1,LG
      NF=NFNG(L)
      IF (NF.LE.0) GO TO 300
      XX=XX+RT(I,L)*PFNG(L)*PR(NF)
      YY=YY-XM(I)*(RT(I,L)-R(I,L))*PFNG(L)*PR(NF)
  300 CONTINUE
      XG (I)=XG (I)+XX
      DPU(I)=DPU(I)+YY
  400 CONTINUE
C
  450 NS=NS+NEQB
      NE=NE+NEQB
```

```
      IF (NE.GT.NEQ) NK=NEQ-NS+1
  500 CONTINUE
C
      DO 600 I=1,NEQ
  600 XT(I)=X2(I)+XC(I)
      IF (NCT.EQ.NCUT) WRITE (8) (X(I),I=1,NEQ),(XT(I),I=1,NEQ)
      RETURN
      END
```

```
      SUBROUTINE OUTPUT
C
      COMMON/ELPAR/ PAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6,N7
      COMMON/EM/ QCCQ(2043)
      COMMON/MISC/ LG,LL,NFNR,NFA,NGM,NAT,NT,NOT,MSTEP
      COMMON/JUNK/ NOS,DT,KK1,KK2,ISP1,ISP2,NSO,NSS,NNS,MDIS,MSTR,
     .             NDISB,NSTRB,NBO,NBS,JUK(190)
      COMMON A(1)
C
C     INPUT SPECIFICATIONS FOR OUTPUT OF RESPONSE TIME HISTORIES
C
      N1=1
      N2=N1+NUMNEL*12
      N3=N2+NEQ
      N4=N3+NUMEL
      CALL INOUT (A(N1),A(N2),A(N3),A(N4),NUMNP,NEQ,NUMEL,NUMNEL)
C
C     PACK RESPONSE TIME HISTORIES IN BLOCKS
C
      DT=NOT*DT
      NOS=NT/NOT
      NDISB=(MTOT-N3-NEQ*2)/(MDIS*2)
      MDISB=(MTOT-N3-9*NOS)/(MDIS*2)
      IF (NDISB.GT.MDISB) NDISB=MDISB
      IF (NDISB.GT.NOS) NDISB=NOS
      NBO=(NOS-1)/NDISB+1
C
      NSTRB=(MTOT-N3-NUMNEL*25)/(MSTR*2)
      MSTRB=(MTOT-N3-9*NOS)/(MSTR*2)
      IF (NSTRB.GT.MSTRB) NSTRB=MSTRB
      IF (NSTRB.GT.NOS) NSTRB=NOS
      NBS=(NOS-1)/NSTRB+1
C
      N4=N3+NEQ
      N5=N4+NEQ
      N6=N5+MDIS*NDISB
C
      N7=N3+NUMNEL
      N8=N7+NUMNEL*12
      N9=N8+NUMNEL*12
      N10=N5+MSTR*NSTRB
      N11=N10+MSTR*NSTRB
      CALL REPACK (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),
     .             A(N3),A(N7),A(N8),A(N9),A(N10),NOS,NEQ,
     .             MDIS,NDISB,NUMNEL,MSTR,NSTRB,NBO,NBS )
C
      MT=0
      NFILE=0
      IF (KK1.EQ.4) NFILE=NFILE+NSO*2
      IF (KK2.EQ.4) NFILE=NFILE+NSS+NNS
      IF (NFILE.EQ.0) GO TO 100
      MT=30
      REWIND MT
      WRITE (MT) NFILE,NOS,DT
  100 N4=N3+NOS
      N5=N4+NOS*8
      N6=N5+MDIS*NDISB
      N7=N5+MSTR*NSTRB
      IF=0
C
```

```
C
C     OUTPUT SELECTED (ABSOLUTE) ACCELERATION TIME HISTORY
C
      REWIND 4
      CALL CUTHIS (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),NUMNEL,
     .            NOS,MDIS,NDISB,NSD,NBD,2,KK1,ISP1,4,12,7,MT,IF)
C
C     OUTPUT SELECTED (RELATIVE) DISPLACEMENT TIME HISTORY
C
      REWIND 4
      CALL CUTHIS (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),NUMNEL,
     .            NOS,MDIS,NDISB,NSD,NBD,1,KK1,ISP1,4,3 ,7,MT,IF)
C
C     OUTPUT SELECTED STRESS TIME HISTORIES OF LINEAR ELEMENTS
C
      REWIND 7
      CALL CUTHIS (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),NUMNEL,
     .            NOS,MDIS,NDISB,NSS,NBD,3,KK2,ISP2,4,3 ,7,MT,IF)
C
C     OUTPUT SELECTED STRESS TIME HISTORIES OF NONLINEAR ELEMENTS
C
      REWIND 2
      CALL CUTHIS (A(N1),A(N2),A(N3),A(N4),A(N5),A(N7),NUMNEL,
     .            NOS,MSTR,NSTRB,NNS,NBS,4,KK2,ISP2,2,10,7,MT,IF)
C
      RETURN
      END


      SUBROUTINE INOUT (ISTR,IDIS,INO,IO,NUMNP,NEQ,NUMEL,NUMNEL)
C
      DIMENSION ISTR(NUMNEL,12),IDIS(NEQ),INO(NUMEL),IO(NUMNP,6)
      COMMON/EM/ NS,ND,LM(24),SA(12,24),KLM(8,24),SSA(8,24),QQQQ(1345)
      COMMON/JUNK/ NOS,OT,KK1,KK2,ISP1,ISP2,NSO,NSS,NNS,MDIS,MSTR,
     .            NDISB,NSTRB,NBD,NBS,KO(4,8),SD(8),NKS(4,8),SND(8),
     .            IC(6),IS(12),NOOF,SIG(12)
C
C     INPUT SPECIFICATIONS FOR OUTPUT RESPONSE TIME HISTORY
C
      REWIND 1
      REWIND 2
      REWIND 4
      REWIND 7
      REWIND 8
C
      READ (8) IO
C
      DO 10 I=1,NEQ
   10 IDIS(I)=0
      DO 20 I=1,NUMNEL
      DO 20 J=1,12
   20 ISTR(I,J)=0
C
C     READ AND PRINT DISPLACEMENT OUTPUT SPECIFICATIONS
C
      L=0
      K=0
      READ (5,1000) KK1,ISP1
  100 READ (5,1000) NP,IC
      IF (NP.GT.0) GO TO 110
      IF (L .EQ.0) GO TO 200
      WRITE (4) KO,SD,L
      K=K+1
      GO TO 200
C
  110 IF (K.EQ.0.AND.L.EQ.0)  WRITE(6,2000)
      WRITE(6,2001) NP,IC
      DO 150 I=1,6
      II=IC(I)
      IF (II.LE.0) GO TO 150
      JJ=IO(NP,II)
      IF (JJ.LE.NEQ) GO TO 120
  115 WRITE(6,4001) NP,II
      GO TO 150
  120 IF (JJ.LE.0) GO TO 115
      L=L+1
      KO(1,L)=6
      KO(2,L)=NP
      KO(3,L)=II
      KO(4,L)=JJ
      SD(  L)=0.
      IDIS(JJ)=1
      IF (L.LT.8) GO TO 150
      WRITE (4) KO,SD,L
      K=K+1
      L=0
  150 CONTINUE
      GO TO 100
C
C     READ AND PRINT STRESS OUTPUT SPECIFICATIONS
C
  200 NSO=K
      WRITE(6,4000) KK1,ISP1
C
      READ (8) INO
      L=0
      LL=0
      K=0
      KK=0
      READ (5,1000) KK2,ISP2
      NREAD=1
      NTYPE=0
      NUMNE=0
C
      DO 600 N=1,NUMEL
      MTYPE=INO(N)
      IF (MTYPE.GT.0) GO TO 205
      MTYPE=-MTYPE
      GO TO 210
  205 NUMNE=NUMNE+1
  210 IF (NTYPE.EQ.MTYPE) GO TO 220
      NTYPE=MTYPE
      NUME=0
  220 NUME=NUME+1
      READ (1) NS,ND,LM,SA
      READ (8) NOOF,SIG
      IF (NREAD.EQ.0) GO TO 300
  225 READ (5,1000) NELTYP,NEL,IS
      IF (NEL.GT.0) GO TO 250
      IF ( L .EQ.0) GO TO 230
      WRITE (4) KO,SD,L
      WRITE (7) KLM,SSA,NO
      K=K+1
```

```
  230 IF (LL .EQ.0) GO TO 700
      WRITE (2) NKS,SND,LL
      KK=KK+1
      GO TO 700
C
  250 IF (N.EQ.1)  WRITE(6,3000)
      WRITE(6,3002) NELTYP,NEL,IS
C
  300 IF (NELTYP.EQ.NTYPE.AND.NEL.EQ.NUME) GO TO 350
      NREAD=0
      GO TO 600
  350 NREAD=1
      IF ((KD(N)) 400,400,500
C
C     STRESSES FOR LINEAR ELEMENTS
C
  400 DO 450 I=1,NS
      II=IS(I)
      IF (II.EQ.0) GO TO 450
      L=L+1
      KD(1,L)=NTYPE
      KD(2,L)=NEL
      KD(3,L)=II
      KD(4,L)=0
      SD(  L)=S(G(II)
      DO 440 J=1,NC
      SSA(L,J)=SA(II,J)
      KLM(L,J)=0
      LMJ=LM(J)
      IF (LMJ.GT.NEQ) GO TO 440
      IF (LMJ.LE. 0 ) GO TO 440
      KLM(L,J)=LMJ
      IDIS(LMJ)=1
  440 CONTINUE
      IF (L.LT.8) GO TO 450
      WRITE (4) KD,SD,L
      WRITE (7) KLM,SSA,ND
      L=0
      K=K+1
  450 CONTINUE
      GO TO 600
C
C     STRESSES FOR NONLINEAR ELEMENTS
C
  500 DO 550 I=1,NS
      II=IS(I)
      IF (II.EQ.0) GO TO 550
      LL=LL+1
      NKS(1,LL)=NTYPE
      NKS(2,LL)=NEL
      NKS(3,LL)=II
      NKS(4,LL)=NUMNE
      SND(  LL)=0.
      LL=LL+1
      NKS(1,LL)=NTYPE
      NKS(2,LL)=NEL
      NKS(3,LL)=II
      NKS(4,LL)=NUMNE
      SND(  LL)=0.
      ISTR(NUMNE,II)= 1
      IF (LL.LT.8) GO TO 550
      WRITE (2) NKS,SND,LL
      LL=0
      KK=KK+1
  550 CONTINUE
  600 CONTINUE
      GO TO 225
C
  700 NSS=K
      NNS=KK
      WRITE(6,4000) KK2,ISP2
      IF (N.GE.NUMEL) GO TO 750
      N=N+1
      DO 720 I=N,NUMEL
      READ (8)
  720 CONTINUE
C
  750 MDIS=0
      MSTR=0
      DO 800 I=1,NEQ
      IF (IDIS(I).EQ.0) GO TO 800
      MDIS=MDIS+1
      IDIS(I)=MDIS
  800 CONTINUE
C
      DO 900 I=1,NUMNEL
      DO 900 J=1,12
      IF (ISTR(I,J).EQ.0) GO TO 900
      MSTR=MSTR+1
      ISTR(I,J)=MSTR
  900 CONTINUE
C
      RETURN
C
 1000 FORMAT (14I5)
 2000 FORMAT (36H1DISPLACEMENT  COMPONENTS  FOR WHICH  /
     .        36H OUTPUT TIME HISTORY IS REQUIRED....  //
     .         5H NODE 4X 23HDISPLACEMENT COMPONENTS /)
 2001 FORMAT (I5,4X,6I4)
 3000 FORMAT (36H1ELEMENT STRESS COMPONENTS FOR WHICH /
     .        36H OUTPUT TIME HISTORY IS REQUIRED....  //
     .        40H  ELEMENT       DESIRED STRESS COMPONENTS /
     .        10H TYPE  NO.    /)
 3002 FORMAT (2I4,5X,12I3)
 4000 FORMAT (/16H OUTPUT TYPE.... ,I2 /
     .        16H PLOT SPACING... ,I2 )
 4001 FORMAT (I5,4X,I4,4X,21HFIXED DOF...NO OUTPUT )
      END


      SUBROUTINE REPACK (ISTR,IDIS,X,X2,XH,X2H,NSS,STR,UPL,STH,UPH,
     .                   NOS,NEQ,MDIS,NOISB,NUMNEL,MSTR,NSTRB,NBD,NBS)
C
      DIMENSION ISTR(NUMNEL,1),IDIS(1),X(NEQ),XH(MDIS,NDISB),
     .          NSS(NUMNEL),STR(NUMNEL,12),UPL(NUMNEL,12),
     .          STH(MSTR,NSTRB),UPH(MSTR,NSTRB),X2(NEQ),X2H(MDIS,NDISB)
C
C     PACK RESPONSE TIME HISTORY IN OPTIMAL AVAILABLE STORAGE BLOCKS
C
      REWIND 3
      REWIND 9
      REWIND 10
```

```
      REWIND 12
      L=0
      K=0
      DO 200 N=1,NOS
      READ (8) X,X2
      L=L+1
      DO 100 I=1,NEG
      II=IDIS(I)
      IF (II) 100,100,90
   90 XM (II,L)=X (I)
      X2M(II,L)=X2(I)
  100 CONTINUE
      IF (L.LT.NOISE) GO TO 200
      WRITE (3)  L,XM
      WRITE (12) L,X2M
      K=K+1
      L=0
  200 CONTINUE
      IF (L) 220,220,210
  210 WRITE (3)  L,XM
      WRITE (12) L,X2M
      K=K+1
  220 IF (NBD.NE.K) NBD=K
C
      L=0
      K=0
      IF (NUMNEL.LE.0) GO TO 420
      DO 400 N=1,NCS
      READ (9) NSS,STR,UPL
      L=L+1
      DO 300 I=1,NUMNEL
      NS=NSS(I)
      DO 250 J=1,NS
      II=ISTR(I,J)
      IF (II) 250,250,240
  240 STM(II,L)=STR(I,J)
      UPM(II,L)=UPL(I,J)
  250 CONTINUE
  300 CONTINUE
      IF (L.LT.NSTPE) GO TO 400
      WRITE (10) L,STM,UPM
      L=0
      K=K+1
  400 CONTINUE
      IF (L) 420,420,410
  410 WRITE (10) L,STM,UPM
      K=K+1
  420 IF (NBS.NE.K) NBS=K
C
      RETURN
      END


      SUBROUTINE OUTHIS (ISTR,IDIS,T,X,XM,UM,NEL,NCS,NOI,NOJ,
     *                   NOB,NNB,KKK,KKI,ISPI,IT,JT,KT,MT,IF)
C
      DIMENSION ISTR(NEL,1),IDIS(1),T(1),X(8,NOS),XM(NOI,NOJ),
     *          UM(NOI,NOJ)
      COMMON/EM/ KLM(8,24),SA(8,24),ND,SM(8)
      COMMON/JUNK/ NPT,DT,NDATA(13),L,K,KO(4,8),XD(8),TM(8),XM(8)
      DATA SM /1H ,1HA,1H2,1HB,1H3,1HC,1H4,1HD/
C
C     OUTPUT RESPONSE TIME HISTORIES ON SPECIFIED DISPLAY MEDIUM
C
C     TAPE IT   INPUT  TAPE     STORE KO(4,8),XD(8),L
C     TAPE JT   INPUT  TAPE     STORE XM(NOI,NOJ),UM(NOI,NOJ),K
C     TAPE KT   INPUT  TAPE     STORE KLM(8,24),SA(8,24),ND
C     TAPE MT   OUTPUT TAPE     STORE IF,KKK,L,KO,XM,X(8,NCS)
C
      IF (NCB.EQ.0) RETURN
C
      DO 900 M=1,NCB
      IF=IF+1
      REWIND JT
      READ (IT) KO,XO,L
      IF (KKK.EQ.3) READ (KT) KLM,SA,ND
      DO 100 I=1,8
      TM(I)=0.
      XM(I)=0.
  100 CONTINUE
C
C     PRINT APPROPRIAT TITLE
C
      GO TO (110,200,130,110) KKI
C
  110 GO TO (111,112,113,114) KKK
  111 WRITE(6,1C01) M,IF
      WRITE(6,1C02) (KO(2,I),KO(3,I),I=1,L)
      GO TO 200
  112 WRITE(6,2C01) M,IF
      WRITE(6,1C02) (KO(2,I),KO(3,I),I=1,L)
      GO TO 200
  113 WRITE(6,3C01) M,IF
      WRITE(6,3C02) (KO(1,I),KO(2,I),KO(3,I),I=1,L)
      GO TO 200
  114 WRITE(6,4C01) M,IF
      WRITE(6,4C02) (KO(1,I),KO(2,I),XD(3,I),I=1,L)
      GO TO 200
C
  130 IF (M.GT.1) GO TO 200
      GO TO (131,132,133,131) KKK
  131 WRITE(6,1C03)
      WRITE(6,1C10)
      GO TO 200
  132 WRITE(6,2C03)
      WRITE(6,2C10)
      GO TO 200
  133 WRITE(6,3C03)
      WRITE(6,3C10)
C
C     ARRANGE RESPONSE TIME HISTORIES IN OUTPUT FORM
C
  200 TT=0.
      N =0
      DO 6CC NB=1,NNB
      GO TO (210,210,220,220) KKK
  210 READ (JT) K,XM
      GO TO 250
  220 READ (JT) K,XM,UM
  250 DO 5CC J=1,K
      N=N+1
      TT=TT+DT
```

```
      MM=-1
      DC 400 I=1,L
C
      GO TO (310,31C,330,340) KKK
C
  310 JJ=KD(4,I)
      II=IOIS(JJ)
      XX=XH(II,J)
      GO TO 350
C
  330 XX=XD(I)
      DO 335 KK=1,NC
      JJ=KLP(I,KK)
      IF (JJ) 335,335,334
  334 II=IOIS(JJ)
      XX=XX+SA(I,KK)*XH(II,J)
  335 CONTINUE
      GO TO 350
C
  340 JJ=KD(3,I)
      NN=KD(4,I)
      II=ISTR(NN,JJ)
      IF (MM) 341,341,342
  341 XX=XH(II,J)
      GO TC 345
  342 XX=UH(II,J)
  345 MM=-1*MM
C
  350 AX=ABS(XX)
      IF (AX-XM(I)) 370,370,360
  360 XM(I)=AX
      TM(I)=TT
  370 X (I,N)=XX
      T ( N )=TT
  400 CONTINUE
C
  500 CONTINUE
  600 CONTINUE
C
      GO TO (610,620,630,640) KKK
C
C     PRINT RESPONSE TIME HISTORY
C
  610 DC 611 N=1,NCS
  611 WRITE(6,1C04) T(N),(X(I,N),I=1,L)
      WRITE(6,1C05) (XM(I),I=1,L)
      WRITE(6,1C06) (TM(I),I=1,L)
      GO TO 900
C
C     PLOT RESPONSE TIME HISTORY
C
  620 GC TO (621,623,625,627) KKK
C
  621 ISO=1
      WRITE(6,1C08) M
      WRITE(6,1C10)
      GO TO 624
  623 ISO=1
      WRITE(6,2C08) M
      WRITE(6,2C10)
  624 WRITE(6,2C11) (KD(2,I),KD(3,I),XM(I),TM(I),I,I=1,L)
      GO TC 629
  625 ISO=1
      WRITE(6,3C08) M
      WRITE(6,3C10)
      WRITE(6,3C11) (KD(1,I),KD(2,I),KD(3,I),XM(I),TM(I),I,I=1,L)
      GO TO 629
  627 ISD=2
      WRITE(6,4C09) M
      WRITE(6,3C10)
      DO 628 I=1,L,2
      II=I+1
      WRITE(6,4011) KD(1,I),KD(2,I),KD(3,I),XM(I),TM(I),SM(I)
  628 WRITE(6,4012) XM(II),TM(II),SM(II)
C
  629 CALL PLOT (X,XM,L,DT,NUS,ISPI,ISO)
      GO TO 900
C
C     PRINT MAXIMUM RESPONSE ONLY
C
  630 GO TO (631,631,635,637) KKK
  631 WRITE(6,1C07) (KD(1,I),KD(2,I),XM(I),TM(I),I=I,L)
      GO TO 900
  635 WRITE(6,3C07) (KD(1,I),KD(2,I),KO(3,I),XM(I),TM(I),I=1,L)
      GO TO 900
  637 DO 638 I=1,L,2
      II=I+1
      WRITE(6,4007) KD(1,I),KD(2,I),KO(3,I),XM(I),TM(I)
  638 WRITE(6,4C08) XM(II),TM(II)
      GO TO 900
C
C     STORE RESPONSE TIME HISTORIES ON OUTPUT TAPE MT
C
  640 IF (KKI.EC.4) WRITE (MT) IF,KKK,L,KD,XM,X
      GO TO 610
C
  900 CONTINUE
      RETURN
C
 1001 FORMAT (5CH1TIME HISTORY FOR SELECTED DISPLACEMENT COMPONENTS ,
     . 5H..... ,I3,37X,8HFILE NC.,I3 //
     .                20X,40HNODE NUMBERS AND DISPLACEMENT COMPONENTS  )
 1002 FORMAT (8H    TIME,2X,8(I8,1H-,I2,X))
 1003 FORMAT (55H1MAXIMUM DISPLACEMENT VALUES FROM DYNAMIC RESPONSE ANAL
     .YSIS // )
 1004 FORMAT (OPF8.3,2X,1P8E12.3)
 1005 FORMAT (/24H MAXIMUM ABSOLUTE VALUES  /10H MAXIMUM    ,1P8E12.3)
 1006 FORMAT (1CH TIME       ,1P8E12.3)
 1007 FORMAT (I6,I12,1PE18.3,E12.3,5X,2HN/ )
 1008 FORMAT (59H1NCRMALIZED PLOT OF DISPLACEMENT RESPONSE TIME HISTORIE
     .S... ,I3 /)
 1010 FORMAT (57H   NODE      DISPLACEMENT      MAXIMUM       TIME AT    PLO
     .T      /58H  NUMBER     COMPONENT         VALUES        MAXIMUM    SYM
     .BOL  )
 2001 FORMAT (5CH1TIME HISTORY FOR SELECTED ACCELERATION COMPONENTS ,
     . 5H..... ,I3,37X,8HFILE NC.,I3 //
     .                20X,40HNODE NUMBERS AND ACCELERATION COMPONENTS  )
 2003 FORMAT (55H1MAXIMUM ACCELERATION VALUES FROM DYNAMIC RESPONSE ANAL
     .YSIS // )
 2008 FORMAT (59H1NCRMALIZED PLOT OF ACCELERATION RESPONSE TIME HISTORIE
     .S... ,I3 /)
 2010 FORMAT (57H   NODE      ACCELERATION      MAXIMUM       TIME AT    PLO
     .T      /58H  NUMBER     COMPONENT         VALUES        MAXIMUM    SYM
     .BOL  )
```

```
 2011 FORMAT (I6,I13,1PE18.3,E12.3,I6)
 3001 FORMAT (46HITIME HISTORIES FOR SELECTED STRESS COMPONENTS ,5H.....
     .      I3,41X,8HFILE NO.,I3 //
     .         2CX,48HELEMENT TYPE - ELEMENT NUMBER - STRESS COMPONENT )
 3002 FORMAT (8H    TIME ,2X,8(I4,2H -,I3,1H-,I2))
 3003 FORMAT (53H1MAXIMUM STRESS VALUES FROM DYNAMIC RESPONSE ANALYSIS
     .  //)
 3007 FORMAT (I6,I6,I10,1PE18.3,E12.3,5X,2HNA)
 3008 FORMAT (53H1NCRMALIZED PLOT OF STRESS RESPONSE TIME HISTORIES...
     .          I3/)
 3010 FORMAT (13H ELEMENT      2X 6HSTRESS 10X 7HMAXIMUM 6X 7HTIME AT 4X
     .4HPLCT /13H TYPE   NUMBER 2X 9HCOMPONENT 7X 7HVALUES   6X 7HMAXIMUM
     .4X 6HSYMBOL )
 3011 FORMAT (I6,I6,I10,1PE18.3,E12.3,I6)
 4001 FORMAT (47H1TIME HISTORIES FOR SELECTED NONLINEAR STRESSES  /
     . 6X,46H AND CORRESPONDING NONLINEAR DEFORMATIONS  .....  I3,
     . 38X,8HFILE NC.,I3 //
     .20X,48HELEMENT TYPE - ELEMENT NUMBER - STRESS COMPONENT  )
 4002 FORMAT (12X,4(12H STRESS      ,12HDEFORMATION )/
     .          8H    TIME,2X,8(I4,2H -,I3,1H-,I2))
 4003 FORMAT (68H1MAXIMUM NONLINEAR STRESSES AND CORRESPONDING NONLINEAR
     . DEFORMATIONS //)
 4007 FORMAT (2I6,I10,1PE18.3,E12.3,2X,6HSTRESS )
 4008 FORMAT (22X,1PE18.3,E12.3,2X,11HDEFORMATION )
 4009 FORMAT (60H1NCRMALIZED PLOT OF NONLINEAR STRESS RESPONSE TIME HIST
     .ORIES /58H AND CORRESPONDING NONLINEAR DEFORMATION TIME HISTORIES.
     ...  I3/)
 4011 FORMAT (I6,I6,I10,1PE18.3,E12.3,5X,A1)
 4012 FORMAT (22X,1PE18.3,E12.3,5X,A1)
      END


      SUBROUTINE PLOT (X,XM,L,DT,NOS,ISP,ISD)
C
      DIMENSION X(8,1),XM(8),PM(8),QM(8),SM(8),IP(8)
      COMMON/EM/ PP(101)
      DATA PM /1H1,1H2,1H3,1H4,1H5,1H6,1H7,1H8/
      DATA QM /1H1,1HA,1H2,1HB,1H3,1HC,1H4,1HO/
      DATA BL,V,P /1H ,1H.,1H+/
C
C     NORMALIZED PLOT OF TIME HISTORIES OF TIME FUNCTIONS
C
      DO 30 I=1,L
      GO TO (10,20) ISD
   10 SM(I)=PM(I)
      GO TO 30
   20 SM(I)=QM(I)
   30 CONTINUE
C
      DO 100 I=1,L
      IF (XM(I)) 50,100,50
   50 XM(I)=50./XM(I)
  100 CONTINUE
      TT=0.
      WRITE(6,2000)
      WRITE(6,2001)
      WRITE(6,2002) TT,P,(V,I=1,24),P,(V,I=1,24),P,(V,I=1,24),P,
     .                      (V,I=1,24),P,TT
C
      K=1
      DO 200 I=2,100
  200 PP(I)=BL
      NOS1=NOS-1
      DO 500 N=1,NOS1
      PP(  1)=V
      PP( 51)=V
      PP(101)=V
      II=ISP
  210 IF (II.LE.0) GO TO 250
      WRITE(6,2003) PP
      II=II-1
      GO TO 210
  250 TT=TT+DT
      DO 300 I=1,L
      XX=XM(I)*X(I,N)
      M=XX
      M=M+51
      IP(I)=M
  300 PP(M)=SM(I)
      IF (K.LT.10) GO TO 320
      PP(  1)=P
      PP( 51)=P
      PP(101)=P
      K=1
      WRITE(6,2002) TT,PP,TT
      GO TO 340
  320 WRITE(6,2003) PP
      K=K+1
C
C     RESET PP(101)
C
  340 DO 360 I=1,L
      M=IP(I)
  360 PP(M)=BL
  500 CONTINUE
C
      TT=TT+DT
      WRITE(6,2002) TT,P,(V,I=1,24),P,(V,I=1,24),P,(V,I=1,24),P,
     .                    (V,I=1,24),P,TT
      WRITE(6,2001)
      RETURN
C
C     *CARRIAGE CONTROL CHACTER 1 SURPRESS SKIP OVER PAGE FOLDS
C
 2000 FORMAT (/56X,8HORDINATE )
 2001 FORMAT (/12HZ  TIME -1.0,21X,4H-0.5,22X,3H0.0,22X,3H0.5,22X,3H1.0,
     .        X,4HTIME )
 2002 FORMAT (1H1,F7.2,2X,101A1,F7.2)
 2003 FORMAT (1H1,5X,101A1)
      END
```

```
      SUBROUTINE TRUSS
C
      COMMON A(1)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMMEL,NEG,NEQG,MBAND,
     .             NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/JUNK/ STR(4),MM,L,K,NTAG,NOYN,SIG(12),EXRA(184)
C
      IF (NPAR(1).EQ.0) GO TO 500
C
      N7=N6+NPAR(3)
      N8=N7+NPAR(3)
      N9=N8+NPAR(3)
      N10=N9+NPAR(3)
      N11=N10+NPAR(3)
      N12=N11+NPAR(4)*4
      IF (N12.GT.MTOT) CALL ERROR (1,N12-MTOT,7HTRUSS  )
      CALL RUSS (A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),A(N9),
     .          A(N10),A(N11),NUMNP,NPAR(4))
      RETURN
C
  500 IF (NTAG.EQ.0) WRITE(6,2002)
      WRITE(6,3002) MM,L,SIG(1),SIG(2)
      NTAG=1
      RETURN
C
 2002 FORMAT (//26HC ....TRUSS MEMBER ACTIONS //
     .           46H0 MEMBER    LOAD          STRESS          FORCE  )
 3002 FORMAT (2I8,F15.5,F15.3)
C
      END



      SUBROUTINE RUSS (ID,X,Y,Z,INO,E,THERM,DEN,AREA,WT,ENPROP,NUMNP,
     .                 NUMNPAR)
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),INO(1),E(1),THERM(1),DEN(1),
     .          AREA(1),WT(1),ENPROP(NUMNPAR,1)
      COMMON/ELPAR/ NPAR(14),NNNNN,NELTYP,NUMEL,NUMMEL,NEQ,NEQG,MBAND,
     .             NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/EM/ LM(24),NO,NS,S(24,24),P(24,4),XM(24),ST(12,24),TT(12,4)
     .          ,QCQQ(984)
      COMMON /JUNK/ EMUL(4,4),I,J,K,L,M,N,II,JJ,KK,MTYPE,TEMP,DX,DY,DZ,
     . XL2,XL,XX,YY,F,FT,FX,FY,FZ,MIN,MAX,NOIF,KKK,TEM,MTYP
C
C     CONTROL INFORMATION AND MEMBER PROPERTIES
C
      NUME=NPAR(2)
      NUMMAT=NPAR(3)
      WRITE (6,2000) NUME,NUMMAT,NUMNPAR
      WRITE (6,2001)
      DO 10 I=1,NUMMAT
      READ  (5,1001) N,E(N),THERM(N),DEN(N),AREA(N),WT(N)
   10 WRITE (6,2002) N,E(N),THERM(N),DEN(N),AREA(N),WT(N)
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ (5,1003)  EMUL
      WRITE (6,2003) EMUL
C
C     READ AND PRINT ELEMENT NONLINEAR PARAMETERS
C
      IF (NUMNPAR.EQ.0) GO TO 50
      WRITE(6,2010)
      DO 30 I=1,NUMNPAR
   30 READ (5,1005) N,(ENPROP(N,J),J=1,4)
      WRITE(6,2011) (N,(ENPROP(N,J),J=1,4),N=1,NUMNPAR)
   50 CONTINUE
C
C     ELEMENT INFORMATION
C
      WRITE (6,2005)
      N=1
  100 READ (5,1004)  M,II,JJ,MTYP,TEM,ININO,KK
      IF (ININO.GT.NUMNPAR) CALL ERROR (3,M,7HTRUSS  )
      IF(KK.EQ.0) KK=1
  120 IF(M.NE.N) GO TO 200
      I=II
      J=JJ
      MTYPE=MTYP
      TEMP=TEM
      NINO=ININO
      KKK=KK
C
C     1. FORM ELEMENT STIFFNESS AND STRESS MATRICES
C
  200 DX=X(I)-X(J)
      DY=Y(I)-Y(J)
      DZ=Z(I)-Z(J)
      XL2=DX*DX+DY*DY+DZ*DZ
      XL=SQRT(XL2)
      XX=E(MTYPE)*AREA(MTYPE)*XL
      ST(1,1)=DX/XL2
      ST(1,2)=DY/XL2
      ST(1,3)=DZ/XL2
      ST(1,4)=-ST(1,1)
      ST(1,5)=-ST(1,2)
      ST(1,6)=-ST(1,3)
C
      DO 300 L=1,6
      YY=ST(1,L)*XX
      DO 250 K=L,6
      S(K,L)=ST(1,K)*YY
  250 S(L,K)=S(K,L)
      ST(1,L)=E(MTYPE)*ST(1,L)
  300 ST(2,L)=AREA(MTYPE)*ST(1,L)
C
C     2. INERTIA AND THERMAL LOADS
C
      F=WT(MTYPE)*XL/2.
      FT=TEMP*THERM(MTYPE)*E(MTYPE)*AREA(MTYPE)
      FX=DX*FT/XL
      FY=DY*FT/XL
      FZ=DZ*FT/XL
C
      DO 350 L=1,4
      TT(2,L)=EMUL(L,4)*FT
      TT(1,L)=TT(2,L)/AREA(MTYPE)
      P(1,L)=EMUL(L,1)*F-EMUL(L,4)*FX
      P(2,L)=EMUL(L,2)*F-EMUL(L,4)*FY
      P(3,L)=EMUL(L,3)*F-EMUL(L,4)*FZ
      P(4,L)=EMUL(L,1)*F+EMUL(L,4)*FX
```

```
      P(5,L)=EMUL(L,2)*F+EMUL(L,4)*FY
  350 P(6,L)=EMUL(L,3)*F+EMUL(L,4)*FZ
      F=DEN(MTYPE)*AREA(MTYPE)*XL/2.
      DO 375 L=1,6
  375 XM(L)=F
C
C     3. FORM LOCATION MATRIX AND COMPUTE BAND WIDTH
C
      DO 400 L=1,3
      LM(L)=ID(I,L)
  400 LM(L+3)=ID(J,L)
C
      NO=6
      NS=2
      CALL WRITET(MBAND,NDIF)
C
C     4. INPUT NONLINEAR ELEMENT PROPERTIES
C
      NN=NUMEL+N
      IF (NINO.EQ.0) GO TO 500
      INO(NN)=1
      WRITE(6,2100)
      STOP
C
  500 INO(NN)=-1
C
C     5. CHECK FOR MORE ELEMENTS
C
      WRITE (6,2004) N,I,J,MTYPE,TEMP,NDIF,NINO
      IF (N.EQ.NUMEL) RETURN
      N=N+1
      I=I+KKK
      J=J+KKK
      IF(N.GT.M) GO TO 100
      GO TO 120
C
 1001 FORMAT (I5,5F10.0)
 1003 FORMAT (4F10.0)
 1004 FORMAT (4I5,1F10.0,2I5)
 1005 FORMAT (I5,4F10.0)
 2000 FORMAT (38H1THREE DIMENSIONAL TRUSS ELEMENTS..... ///
     . 25H NUMBER OF TRUSS MEMBERS= I5//
     1 25H NUMBER OF DIFF. MEMBERS= I5//
     2 25H NUMBER OF NPAR SETS     = I5)
 2001 FORMAT (///1X,4HTYPE,14X,1HE,10X,5HALPHA,12X,3HDEN,11X,4HAREA
     1 11X,4HWT/L )
 2002 FORMAT (I5,5E15.7)
 2003 FORMAT (///30H ELEMENT LOAD MULTIPLIERS...../12X,1HA,14X,1HB,14X,
     .  1HC,14X,1HD /6H X-DIR 4E15.6/6H Y-DIR 4E15.6/6H Z-DIR 4E15.6/
     .  6H  TEMP 4E15.6 )
 2004 FORMAT (4I6,F10.2,2I7)
 2005 FORMAT (///42H1    N     I      J  TYPE       TEMP   BAND 7H    IND)
 2010 FORMAT (1H1,35HTRUSS ELEMENT NONLINEAR PARAMETERS..... //
     .  8H NL PAR. 4X 8HAXIAL-C  4X 8HAXIAL-T  4X 8HPLASTIC  4X
     .  8HPLASTIC /8H    NO  4X 8H   PU    4X 8H   PU    4X 8HC-STIF
     ,  4X 8HT STIF  /)
 2011 FORMAT (I8,4E12.3)
 2100 FORMAT (1H1,65HNONLINEAR TRUSS ELEMENT IS NOT READY YET....EXECUTI
     .ON TERMINATED. )
      END
```

```
      SUBROUTINE BEAM
C
      COMMON A(1)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMMEL,NEQ,NEQG,MBAND,
     .               NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/JUNK/ STR(4),MM,L,K,NTAG,NOIN,SIG(12),EXRA(184)
C
      IF (NPAR(1).EQ.0) GO TO 500
      N7=N6+NPAR(5)
      N8=N7+NPAR(5)
      N9=N8+NPAR(5)
      N10=N9+NPAR(4)*12
      N11=N10+NPAR(3)*6
      N12=N11+NPAR(6)*12
      IF (N12.GT.MTOT) CALL ERROR (1,N12-MTOT,7HBEAM    )
      CALL TEAM (NPAR(2),NPAR(3),NPAR(4),NPAR(5),NPAR(6),
     .           A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),
     .           A(N9),A(N10),A(N11),NUMNP,MBAND,NUMEL)
C
      RETURN
C
  500 IF(NTAG.EQ.0) WRITE (6,2002)
      WRITE(6,3002) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
C
 2002 FORMAT(/25H0 ....BEAM FORCES AND MOMENTS//
     . 10H0BEAM LOAD 5X 5HAXIAL  2(7X,5HSHEAR),5X 7HTORSION
     .  2(5X,7HBENDING)/ 10H NO.   NO.  8X  2HP1  10X  2HR2  10X
     .  2HR3 10X 2HM1 10X 2HM2 10X 2HM3)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
C
      END
```

```
      SUBROUTINE TEAM (NBEAM,NUMETP,NUMFIX,NUMMAT,NUMNPAR,ID,X,Y,Z,IND,
     .                 E,G,RO,SFI,COPROP,ENPROP,NUMNP,MBAND,NUMEL)
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),IND(1),E(1),G(1),RO(1),
     .          SFI(NUMFIX,1),COPROP(NUMETP,1),ENPROP(NUMNPAR,1),
     .          STIF(72),LMS(12),EC(12),T(3,3)
      COMMON/EM/ LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .           SF(12,4),ENPAR(24),S(12,12),F(48),TTT(72)
      COMMON/JUNK/ LC(6),JK(6),MELTYP,DL,MATTYP,ILC(4),TS(2,2),LS(4),
     .           FMUL(3,4)
      EQUIVALENCE (STIF,LM),(LMS,TTT(1)),(EC,TTT(13)),(T,TTT(25))
C
C     FORMS 3-D BEAM STIFFNESS AND STRESS ARRAYS
C
      WRITE (6,2005) NBEAM,NUMETP,NUMFIX,NUMMAT,NUMNPAR
      DO 5 I=1,1346
    5 STIF(I)=0.
C
C     READ AND PRINT MATERIAL PROPERTY DATA
C
      WRITE (6,2001)
      DO 10 I=1,NUMMAT
      READ  (5,1001) N,E(N),G(N),RO(N)
      WRITE (6,2002) N,E(N),G(N),RO(N)
```

```
   10 G(N)=C.5*E(N)/(1.+G(N))
C
C     READ AND PRINT GEOMETRIC PROPERTIES OF COMMON ELEMENTS.
C
      WRITE (6,2003)
      DO 30 I=1,NUMETP
      READ  (5,1002) N,(COPROP(N,J),J=1,6)
      IF((COPROP(N,1).NE.0.0).AND.(COPROP(N,4).NE.0.0).AND.
     1   (COPROP(N,5).NE.0.0).AND.(COPROP(N,6).NE.0.0))  GO TO 20
      WRITE (6,2013)
      CALL EXIT
   20 WRITE (6,2004) N,(COPROP(N,J),J=1,6)
   30 CONTINUE
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ  (5,1006) ((EMUL(I,J),J=1,4),I=1,3)
      WRITE (6,2008) ((EMUL(I,J),J=1,4),I=1,3)
C
C     READ AND PRINT FIXED END FORCES IN LOCAL COORDINATES
C
      IF(NUMFIX .EQ. 0) GO TO 56
      WRITE (6,2010)
      DO 55 I=1,NUMFIX
      READ  (5,1005) N,(SFT(N,J),J=1,12)
   55 WRITE (6,2011) N,(SFT(N,J),J=1,12)
   56 CONTINUE
C
C     READ AND PRINT ELEMENT NONLINEAR PARAMETERS
C
      IF (NUMNPAR.EQ.0) GO TO 59
      WRITE(6,2020)
      DO 58 I=1,NUMNPAR
   58 READ (5,1007) N,(ENPROP(N,J),J=1,12)
      WRITE(6,2021) (N,(ENPROP(N,J),J=1,12),N=1,NUMNPAR)
   59 CONTINUE
C
C     READ AND PRINT ELEMENT DATA. GENERATE MISSING INPUT.
C
      WRITE (6,4000)
      L=0
   60 KKK=0
      READ (5,3000) INEL,INI,INJ,INK,IMAT,IMEL,ILC,INELKI,INELKJ,INC,IDD
      IF (IDD.GT.NUMNPAR) CALL ERROR (3,INEL,7HBEAM    )
      IF (INEL.NE.1) GO TO 15
      NI=INI
      NJ=INJ
      NK=INK
   15 IF (INC.EQ.0) INC=1
   65 L=L+1
      KKK=KKK+1
      ML=INEL-L
      IF (ML)  66,67,68
   66 CALL ERROR (4,INEL,7HBEAM    )
   67 NEL=INEL
      NI=INI
      NJ=INJ
      NK=INK
      MATTYP=IMAT
      MELTYP=IMEL
      DO 90 I=1,4
   90 LC(I)=ILC(I)
      NEKODI=INELKI
      NEKODJ=INELKJ
      NIND=IDD
      GO TO 69
   68 NEL=INEL-ML
      NI=IN+KKK*INCR
      NJ=JN+KKK*INCR
   69 CONTINUE
      WRITE (6,4001) NEL,NI,NJ,NK,MATTYP,MELTYP,LC,NEKODI,NEKODJ,NIND
C
   74 DX=X(NJ)-X(NI)
      DY=Y(NJ) Y(NI)
      DZ=Z(NJ)-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF(DL) 75,75,76
   75 CALL ERROR (5,NEL,7HBEAM    )
C
C     FORM GLOBAL TO LOCAL COORDINATE TRANSFORMATION.
C
   76 T(1,1)=DX/DL
      T(1,2)=DY/DL
      T(1,3)=DZ/DL
C
      A1=X(NJ)-X(NI)
      A2=Y(NJ)-Y(NI)
      A3=Z(NJ)-Z(NI)
      B1=X(NK)-X(NI)
      B2=Y(NK)-Y(NI)
      B3=Z(NK)-Z(NI)
      AA=A1*A1+A2*A2+A3*A3
      AB=A1*B1+A2*B2+A3*B3
      U1=AA*B1 AB*A1
      U2=AA*B2- AB*A2
      U3=AA*B3- AB*A3
      UU=U1*U1+U2*U2+U3*U3
      UU=SQRT(UU)
      IF (UU.GT.0.) GO TO 77
      CALL ERROR (6,NEL,7H BEAM   )
   77 T(2,1)=U1/UU
      T(2,2)=U2/UU
      T(2,3)=U3/UU
      T(3,1)=T(1,2)*T(2,3)-T(1,3)*T(2,2)
      T(3,2)=T(1,3)*T(2,1)-T(1,1)*T(2,3)
      T(3,3)=T(1,1)*T(2,2)-T(1,2)*T(2,1)
C
C     CHECK IF NEW STIFFNESS NEEDED
C
      IF (NEL.EQ.1) GO TO 80
      IF  (ABS(DS-DL).GT.DL/100.)  GO TO 80
      IF ((MT.NE.MATTYP).OR.(ME.NE.MELTYP)) GO TO 80
      IF ((JK(1).NE.NEKODI).OR.(JK(2).NE.NEKODJ)) GO TO 80
      DO 78 I=1,4
      IF (LS(I).NE.LC(I)) GO TO 80
   78 CONTINUE
      DO 79 I=1,2
      DO 79 J=1,2
      IF (ABS(TS(I,J)-T(I,J)).GT.ABS(T(I,J)/100.)) GO TO 80
   79 CONTINUE
      GO TO 150
C
   80 DS=DL
      MT=MATTYP
```

```
      ME=MELTYP
      DO 81 I=1,2
      DO 81 J=1,2
   81 TS(I,J)=T(I,J)
      DO 82 I=1,4
   82 LS(I)=LC(I)
      JK(1)=NEKCOI
      JK(2)=NEKCOJ
C
C     FORM NEW STIFFNESS
C
      CALL NEWBM (E,G,RO,COPROP,SFT,NUMFIX,NUMETP)
C
C     FORM ELEMENT LOCATION MATRIX   .
C
  150 DO 170 M=1,6
      LM(M)=ID(NI,M)
      LM(M+12)=0
      LM(M+18)=0
  170 LM(M+6)=ID(NJ,M)
      NS=12
      ND=12
C
C     TRANSFORM TO MASTER DEGREES OF FREEDOM
C
      CALL SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITET (MBAND,NOIF)
C
C     SET NONLINEAR ELEMENT INDICATOR AND STORE NONLINEAR INFORMATION
C
      NN=NL+FL+NEL
      IF (NINO.LE.0) GO TO 200
      IND(NN)=2
      DO 190 I=1,12
  190 ENPAR(I)=ENPRCP(NINO,I)
      MTYPE=2
      WRITE (3) MTYPE,LM,ND,NS,ASA,SA,S,ENPAR,TT
      GO TO 250
  200 IND(NN)=-2
C
C     CHECK FOR LAST ELEMENT
C
  250 IF (NBEAM-NEL) 66,500,260
  260 IF (ML.GT.0)  GO TO 65
      IN=INI
      JN=INJ
      INCN=INC
      GO TO 60
  500 RETURN
C
 1001 FORMAT(I5,3F10.0)
 1002 FORMAT(I5,6F10.0)
 1005 FORMAT(I5,6F10.0/F15.0,5F10.0)
 1006 FORMAT (4F10.0)
 1007 FORMAT (I5,4F10.0/8F10.0)
 2001 FORMAT (24H1MATERIAL PROPERTIES.... ///
     .        54H MATERIAL    YOUNG S      POISSON S              MASS       /
     .        54H  NUMBER     MODULUS        RATIO             DENSITY      /)
 2002 FORMAT(1H ,I5,3X,F12.0,F14.5,F14.5)
 2003 FORMAT (////32H BEAM GEOMETRIC PROPERTIES....  //
     1   48H ELEMENT        AREA           AREA            AREA
     2  60H INERTIA       INERTIA       INERTIA
     3 / 48H   TYPE              X              Y               Z
     4                    30H     X              Y               Z             )
 2004 FORMAT(1H ,I5,2X,6E12.3)
 2005 FORMAT(30H1.......THREE DIMENSIONAL BEAM ELEMENTS///
     .   36H  NUMBER OF BEAMS                   =,I5//
     .   36H  NUMBER OF GEOMETRIC PROPERTY SETS=,I5//
     .   36H  NUMBER OF FIXED END FORCE SETS    =,I5//
     .   36H  NUMBER OF MATERIALS               =,I5//
     .   36H  NUMBER OF BEAM N/L PROPERTY SETS =,I5)
 2006 FORMAT(///30H ELEMENT LOAD MULTIPLIERS...../12X,1HA,14X,1HB,14X,
     .   1HC,14X,1HD/6H X-DIR 4E15.6/6H Y-DIR 4E15.6/6H Z-DIR 4E15.6// )
 2010 FORMAT (1H1
     1  30X40H FIXED END FORCES IN LOCAL COORDINATES
     2//53H TYPE    NODE          FORCE X      FORCE Y      FORCE Z
     3                     35H MOMENT X      MOMENT Y      MOMENT Z          )
 2011 FORMAT(1H ,I3,6X,1HI,3X,6F12.3/1H ,9X,1HJ,3X,6F12.3/)
 2013 FORMAT(1HC/
     1  60H SECTION PROPERTIES OTHER THAN SHEAR AREAS MAY NOT BE SPECIF
     2  34HIED AS ZERO. EXECUTION TERMINATED.)
 2020 FORMAT (1H1/30X 33HBEAM ELEMENT NONLINEAR PARAMETERS //
     1   8H   NPAR   4X 8HAXIAL      4X 8HMOMENT     4X 8HMOMENT    4X
     2   8H AXIAL    20X 24HYIELD FUNCTION CONSTANTS /
     3   8H    NC    6X 2HP0 9X 4HMO/Y 8X 4HMO/Z 9X 2HTO 8X 2HA0 6X 2HA1
     4   6X 2HA2 6X 2HA3 6X 2HB0 6X 2HB1 6X 2HB2 6X 2HB3  /)
 2021 FORMAT (I5,3X,4E12.3,8F8.3)
 3000 FORMAT (10I5,2I6,1X,I5)
 4000 FORMAT (23H1BEAM ELEMENT DATA.....  //
     .  5H BEAM 5X 5HNODES 5X 5H MATL 5H GEOM 5X 10HELEM LOADS 4X    10X
     .  12H END CODES    5X 5H L/NL /
     .                    5H  NO        4X 1HI 4X 1HJ 4X 1HK 5H  NO   5H  NO
     .  4X 1HA 4X 1HB 4X 1HC 4X 1HD 9X 1HI 9X 1HJ 5X 5H   IND )
 4001 FORMAT (10I5,2I10)
 4004 FORMAT(1H ,31HNODAL POINT NUMBERS FOR ELEMENT,I5,30HARE IDENTICAL.
     1 EXECUTION TERMINATED.)
C
      END
```

```
      SUBROUTINE NEWBM(E,G,RO,COPROP,SFT,NUMFIX,NUMETP)
C
      DIMENSION E(1),G(1),RO(1),COPROP(NUMETP,1),SFT(NUMFIX,1)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .   SF(12,4),ENPAR(24),S(12,12),F(48),LMS(12),EC(12),T(3,3)
      COMMON/JUNK/ LC(4),JK(6),MELTYP,OL,MATTYP,ILC(4),TS(2,2),LS(4),
     .            EMUL(3,4),R(12),C(12),EL(3,4)
C
C     FORM NEW BEAM STIFFNESS
C
      DO 5 I=1,144
    5 S(I)=0.
      AX=COPROP(MELTYP,1)
      AY=COPROP(MELTYP,2)
      AZ=COPROP(MELTYP,3)
      AAX=COPROP(MELTYP,4)
      AAY=COPROP(MELTYP,5)
      AAZ=COPROP(MELTYP,6)
      SHFY=0.0
```

```
      SHFZ=C.0
      ZY=E(MATTYP)/(DL*DL)
      EIY=ZY*AAY
      EIZ=ZY*AAZ
      IF(AY.NE.0.0) SHFY=6.*EIZ/(G(MATTYP)*AY)
      IF(AZ.NE.C.0) SHFZ=6.*EIY/(G(MATTYP)*AZ)
      COMMY=EIY/(1.+2.*SHFZ)
      COMMZ=EIZ/(1.+2.*SHFY)
C
C     FIXED END FORCES IN LOCAL COORDS
C
      DO 73 N=1,4
      M=LC(N)
      IF (M.GT.C)  GO TO 71
      DO 70 I=1,12
   70 SF(I,N)=0.
      GO TO 73
   71 DO 72 I=1,12
   72 SF(I,N)=SFT(M,I)
   73 CONTINUE
C
C     ELEMENT LOCAL FORCES DUE TO GRAVITIES IN 3 DIRECTIONS
C
      DO 75 I=1,12
   75 EL(I)=0.
      PCM=RC(MATTYP)*AX
      DO 80 J=1,4
      DO 80 I=1,3
      XX=0.C
      DO 77 K=1,3
   77 XX=XX+PCM*T(I,K)*EMUL(K,J)
   80 EL(I,J)=XX
C
      DO 100 J=1,4
      K=1
      DO 100 I=1,12,6
      K=-1*K
      SF(I  ,J)=SF(I  ,J)-EL(1,J)*DL/2.0
      SF(I+1,J)=SF(I+1,J)-EL(2,J)*DL/2.0
      SF(I+2,J)=SF(I+2,J)-EL(3,J)*DL/2.0
      SF(I+4,J)=SF(I+4,J)-K*EL(3,J)*DL*DL/12.0
      SF(I+5,J)=SF(I+5,J)+K*EL(3,J)*DL*DL/12.0
  100 CONTINUE
C
C     FORM ELEMENT STIFFNESS IN LOCAL COORDINATES
C
      S(1,1)= E(MATTYP)* AX/DL
      S(4,4)= G(MATTYP)*AAX/DL
      S(2,2)= COMMZ*12./DL
      S(3,3)= COMMY*12./DL
      S(5,5)= COMMY* 4.*DL*(1.+0.5*SHFZ)
      S(6,6)= COMMZ* 4.*DL*(1.+0.5*SHFY)
      S(2,6)= COMMZ* 6.
      S(3,5)=-COMMY* 6.
      DO 102 I=1,6
      J=I+6
  102 S(J,J)=S(I,I)
      DO 104 I=1,4
      J=I+6
  104 S(I,J)=-S(I,I)
      S(6,12)= S(6,6)*(1.-SHFY)/(2.+SHFY)
      S(5,11)= S(5,5)*(1.-SHFZ)/(2.+SHFZ)
      S(2,12)= S(2,6)
      S(6, 8)=-S(2,6)
      S(8,12)=-S(2,6)
      S(3,11)= S(3,5)
      S(5, 9)=-S(3,5)
      S(9,11)= S(3,5)
      DO 106 I=2,12
      K=I-1
      DO 106 J=1,K
  106 S(I,J)=S(J,I)
C
C     MODIFY STIFFNESS AND FIXED END FORCES FOR ZERO END CONDITIONS
C
      IF ((JK(1)+JK(2)).EQ.0) GO TO 145
      DO 140 K=1,2
      KK=JK(K)
      KD=100000
      I1=6*(K-1)+1
      I2=I1+5
      DO 140 I=I1,I2
      IF (KK.LT.KD)  GO TO 140
      SII=S(I,I)
      DO 125 N=1,12
  125 R(N)=S(I,N)
      DO 130 M=1,12
      C(M)=S(M,I)/SII
      DO 130 N=1,12
  130 S(M,N)=S(M,N)-C(M)*R(N)
      DO 135 N=1,4
      SFI=SF(I,N)
      DO 135 M=1,12
  135 SF(M,N)=SF(M,N)-C(M)*SFI
  136 KK=KK-KD
  140 KD=KD/10
  145 CONTINUE
C
C     PERFORM LOCAL TO GLOBAL TRANSFORMATION
C
      DO 31 I=1,288
   31 SA(I)=0.
      DO 150 LA=1,10,3
      LB=LA+2
      DO 150 MA=1,10,3
      MB=MA-1
      DO 150 I=LA,LB
      DO 150 JM=1,3
      J=JM+MB
      XX=0.
      DO 151 K=1,3
  151 XX=XX+S(I,K+MB)*T(K,JM)
  150 SA(I,J)=XX
C
C     ELEM STIFF ASA(12,12) AND FIXED END FORCES PF(12) IN GLOBAL COORDS
C
      DO 32 I=1,576
   32 ASA(I)=0.
      DO 160 LA=1,10,3
      LB=LA-1
      DO 160 MA=1,10,3
      MD=MA+2
      DO 160 IL=1,3
      I=IL+LB
```

```
      DO 160 J=MA,MB
      XX=0.
      DO 161 K=1,3
  161 XX=XX+T(K,IL)*SA(K+LB,J)
  160 ASA(I,J)=XX
C
      DO 165 LA=1,10,3
      LB=LA-1
      DO 165 IL=1,3
      I=IL+LB
      DO 165 N=1,4
      XX=0.
      DO 162 K=1,3
  162 XX=XX-T(K,IL)*SF(K+LB,N)
  165 RF(I,N)=XX
C
C     FORM MASS MATRIX
C
      XXM =ROM*CL/2.0
      XXMX=XXM*(AAX/AX)
      XXMY=XXM*(DL**2/210.+AAY/(AX*5.0))
      XXMZ=XXM*(DL**2/210.+AAZ/(AX*5.0))
      DO 180 M=1,3
      RXM=XXMX*T(1,M)**2+XXMY*T(2,M)**2+XXMZ*T(3,M)**2
      XM(M   )=XXM
      XM(M+3)=RXM
      XM(M+6)=XXM
  180 XM(M+9)=RXM
      RETURN
      END
```

```
      SUBROUTINE SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1)
      COMMON/EM/ LM(24),ND,NS,S(24,24),P(24,4),XM(24),SA(12,24),TT(12,4)
     *           ,ENPAR(24),SS(12,12),F(48),LMS(12),EC(12),T(3,3)
C
C     PERFORMS SLAVE...MASTER DISPLACEMENT TRANSFORMATION
C
      DO 10 I=1,12
      LMS(I)=0
   10 EC(I)=0.
      KK=0
      DO 100 NF=1,12,6
      NOD=NI
      IF (NF.EQ.7) NOD=NJ
      DO 80 K=1,3
      I=K+NF-1
      IF (LM(I).GE.0) GO TO 80
      LMS(I)=1
      M=-LM(I)
      LM(I)=ID(M,K)
      IF (K-2) 35,40,45
   35 D1=-(Y(NOD)-Y(M))
      D2=   Z(NOD)-Z(M)
      LM(ND+1)=ID(M,6)
      LM(ND+2)=ID(M,5)
      GO TO 50
   40 D1=-(Z(NOD)-Z(M))
      D2=   X(NOD)-X(M)
      LM(ND+1)=ID(M,4)
      LM(ND+2)=ID(M,6)
      GO TO 50
   45 D1=-(X(NOD)-X(M))
      D2=   Y(NOD)-Y(M)
      LM(ND+1)=ID(M,5)
      LM(ND+2)=ID(M,4)
   50 KK=KK+1
      EC(KK)=D1
      KK=KK+1
      EC(KK)=D2
C
C     TRANSFORMATION...ARRAYS INCREASE IN SIZE
C
      DO 60 II=1,ND
      S(ND+1,II)=S(I,II)*D1
      S(ND+2,II)=S(I,II)*D2
      XM(ND+1)=XM(I)*D1*D1
      XM(ND+2)=XM(I)*D2*D2
      S(II,ND+1)=S(II,I)*D1
      S(II,ND+2)=S(II,I)*D2
      DO 55 J=1,4
      P(ND+1,J)=P(I,J)*D1
      P(ND+2,J)=P(I,J)*D2
   55 CONTINUE
   60 CONTINUE
      DO 70 II=1,NS
      SA(II,ND+1)=SA(II,I)*D1
   70 SA(II,ND+2)=SA(II,I)*D2
C
      S(ND+1,ND+1)=S(I,I)*D1**2
      S(ND+2,ND+2)=S(I,I)*D2**2
```

```
      S(ND+1,ND+2)=S(1,1)*D1*D2
      S(ND+2,ND+1)=S(ND+1,ND+2)
      ND=ND+2
   80 CONTINUE
C
C     SET ROTATIONS
C
      DO 90 J=1,3
      K=NF+J+2
      IF(LM(K).GE.C)  GO TO 90
      M=-LM(K)
      LM(K)=ID(M,J+3)
   90 CONTINUE
C
  100 CONTINUE
C
      RETURN
      END
```

```
      SUBROUTINE CBEAM
C
      COMMON A(1)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMMEL,NEQ,NEQG,MBAND,
     .              NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
     .COMMON/JUNK/ STR(4),MM,L,K,NTAG,NDYN,SIG(12),EXPA(184)
C
      IF(NPAR(1).EQ.0) GO TO 500
C
      N7=N6+NPAR(5)
      N8=N7+NPAR(5)
      N9=N8+NPAR(5)
      N10=N5+NPAR(4)*12
      N11=N10+NPAR(3)*6
      N12=N11+NPAR(6)
      N13=N12+NPAR(7)*12
      IF (N13.GT.MTOT) CALL ERROR (1,N13-MTOT,7HCBEAM  )
      CALL CTEAM (NPAR(2),NPAR(3),NPAR(4),NPAR(5),NPAR(6),NPAR(7),
     .            A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),A(N7),A(N8),
     .            A(N9),A(N10),A(N11),A(N12),NUMNP,MBAND,NUMEL)
      RETURN
C
  500 IF(NTAG.EQ.0) WRITE (6,2002)
      WRITE(6,3002) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
C
 2002 FORMAT(/36H0 ....CURVED BEAM FORCES AND MOMENTS//
     . 10H0BEAM LOAD 5X 5HAXIAL  2(7X,5HSHEAR),5X 7HTORSION
     .  2(5X,7HBENDING)/ 10H  NO.  NO.  8X  2HR1  10X  2HR2  10X
     .  2HR3 10X 2HM1 10X 2HM2 10X 2HM3)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
C
      END
```

```
      SUBROUTINE CTEAM (NCBEAM,NUMETP,NUMFIX,NUMMAT,NUMRAD,NUMPAR,
     .                  ID,X,Y,Z,INC,E,G,RO,SFT,COPROP,RAD,ENPROP,
     .                  NUMNP,MBAND,NUMEL)
C
C     FORM 3-D CURVED BEAM STIFFNESS AND STRESS-DISP. ARRAYS
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),INC(1),E(1),G(1),RO(1),
     .          SFT(NUMFIX,1),COPROP(NUMETP,1),RAD(1),ENPROP(NUMNP/P,1),
     .          STIF(722),LMS(12),EC(12),TI(3,3),TJ(3,3)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .          SF(12,4),LNPAR(24),S(12,12),F(48),TTT(72)
      COMMON/JUNK/ LC(4),JK(6),MELTYP,DL,MATTYP,BETA,B,CP,D,FP,AREA,
     .             EMUL(3,4),ILC(4),LS(4)
      EQUIVALENCE (STIF,LM),(LMS,TTT(1)),(EC,TTT(13)),(TI,TTT(25)),
     .            (TJ,TTT(34))
C
      WRITE (6,2005) NCBEAM,NUMETP,NUMFIX,NUMMAT,NUMRAD,NUMPAR
      DO 5 I=1,1348
    5 STIF(I)=0.
C
C     READ AND PRINT RADIUS OF CURVATURE DATA
C
      WRITE (6,2000)
```

```
      DO 8 I=1,NUMRAD
      READ (5,1000) N,RAD(N)
      IF (RAD(N).NE.0.0) GO TO 7
      WRITE (6,2014)
      CALL EXIT
    7 WRITE (6,2015) N,RAD(N)
    8 CONTINUE
C
C     READ AND PRINT MATERIAL PROPERTY DATA
C
      WRITE (6,2001)
      DO 10 I=1,NUMPA
      READ  (5,1001) N,E(N),G(N),RO(N)
      WRITE (6,2002) N,E(N),G(N),RO(N)
   10 G(N)=0.5*E(N)/(1.+G(N))
C
C     READ AND PRINT CROSS SECTIONAL PROPERTIES
C
      WRITE (6,2003)
      DO 30 I=1,NUMETP
      READ  (5,1002) N,(COPROP(N,J),J=1,6)
      IF((COPROP(N,1).NE.0.0).AND.(COPROP(N,4).NE.0.0).AND.
     1   (COPROP(N,5).NE.0.0).AND.(COPROP(N,6).NE.0.0))  GO TO 20
      WRITE (6,2013)
      CALL EXIT
   20 WRITE (6,2004) N,(COPROP(N,J),J=1,6)
   30 CONTINUE
C
C     ELEMENT LOAD MULTIPLIERS
C
      READ  (5,1004) ((EMUL(I,J),J=1,4),I=1,3)
      WRITE (6,2006) ((EMUL(I,J),J=1,4),I=1,3)
C
C     READ AND PRINT FIXED END FORCES IN LOCAL COORDINATES
C
      IF(NUMFIX .EQ. 0) GO TO 56
      WRITE (6,2010)
      DO 55 I=1,NUMFIX
      READ  (5,1005) N,(SFT(N,J),J=1,12)
   55 WRITE (6,2011) N,(SFT(N,J),J=1,12)
   56 CONTINUE
C
C     READ AND PRINT ELEMENT NONLINEAR PARAMETERS
C
      IF (NUMPAR.EQ.0) GO TO 59
      WRITE(6,2020)
      DO 58 I=1,NUMPAR
   58 READ (5,1007) N,(ENPROP(N,J),J=1,12)
      WRITE(6,2021) (N,(ENPROP(N,J),J=1,12),N=1,NUMPAR)
   59 CONTINUE
C
C     READ AND PRINT ELEMENT DATA FOR EACH ELEMENT
C
      WRITE (6,4000)
      L=0
   60 READ (5,3000) INEL,INI,INJ,INK,IMAT,IMEL,ILC,INELKI,INELKJ,IRAD,IQ
      IF (IQ.GT.NUMPAR) CALL ERROR (3,INEL,7HCBEAM  )
      L=L+1
      ML=INEL-L
      IF (ML)  66,67,66
   66 CALL ERROR (4,INEL,7HCBEAM  )
   67 NEL=INEL
      NI=INI
      NJ=INJ
      NK=INK
      MATTYP=IMAT
      MELTYP=IMEL
      DO 68 I=1,4
   68 LC(I)=ILC(I)
      NEKODI=INELKI
      NEKODJ=INELKJ
      NR=IRAD
      NINO=IQ
      WRITE (6,4001) NEL,NI,NJ,NK,MATTYP,MELTYP,LC,NEKODI,NEKODJ,NR,NINO
C
C     FORM GLOBAL TO LOCAL COORDINATE TRANSFORMATION.
C
   74 DX=X(NJ)-X(NI)
      DY=Y(NJ)-Y(NI)
      DZ=Z(NJ)-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF(DL) 75,75,76
   75 CALL ERROR (5,NEL,7HCBEAM  )
   76 RA=RAD(NR)
      ARGU=DL/(2.0*RA)
      BETA=2.0*ASIN(ARGU)
      B =SIN(BETA)
      DP=COS(BETA)
      D =1.0-DP
      BX=X(NI)-X(NK)
      BY=Y(NI)-Y(NK)
      BZ=Z(NI)-Z(NK)
      BB=BX*BX+BY*BY+BZ*BZ
      BB=SQRT(BB)
      IF (BB.GT.0.0) GO TO 77
      CALL ERROR (6,NEL,7HCBEAM  )
   77 CX=DY*BZ-DZ*BY
      CY=DZ*BX-DX*BZ
      CZ=DX*BY-DY*BX
      CC=CX*CX+CY*CY+CZ*CZ
      CC=SQRT(CC)
      BX=BX/BB
      BY=BY/BB
      BZ=BZ/BB
      CX=CX/CC
      CY=CY/CC
      CZ=CZ/CC
      AX=BY*CZ-BZ*CY
      AY=BZ*CX-BX*CZ
      AZ=BX*CY-BY*CX
C
      TI(1,1)=BX
      TI(1,2)=BY
      TI(1,3)=BZ
      TI(2,1)=BX
      TI(2,2)=BY
      TI(2,3)=BZ
      TI(3,1)=CX
      TI(3,2)=CY
      TI(3,3)=CZ
C
      TJ(1,1)=AX*DP-BX*B
      TJ(1,2)=AY*DP-BY*B
      TJ(1,3)=AZ*DP-BZ*B
```

```
      TJ(2,1)=AX*B+BX*OP
      TJ(2,2)=AY*B+BY*OP
      TJ(2,3)=AZ*B+BZ*OP
      TJ(3,1)=CX
      TJ(3,2)=CY
      TJ(3,3)=CZ
C
C     CHECK IF NEW STIFFNESS NEEDED
C
      IF (NEL.EQ.1) GO TO 80
      IF (MR.NE.NR)  GO TO 80
      IF  (ABS(DS-DL).GT.DL/100.)  GO TO 80
      IF ((MT.NE.MATTYP).OR.(ME.NE.MELTYP)) GO TO 80
      IF ((JK(1).NE.NEKODI).OR.(JK(2).NE.NEKODJ)) GO TO 80
      DO 79 I=1,4
      IF (LS(I).NE.LC(I)) GO TO 80
   79 CONTINUE
      GO TO 100
C
   80 DS=DL
      MT=MATTYP
      ME=MELTYP
      MR=NR
      DO 85 I=1,4
   85 LS(I)=LC(I)
      JK(1)=NEKODI
      JK(2)=NEKODJ
C
C     FORM NEW CURVED BEAM STIFFNESS
C
      CALL NEWCEM (E,G,RO,COPROP,SFT,RAD,NUMFIX,NUMETP)
C
  100 CALL TRANSFM (RO,COPROP,NUMETP)
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 170 M=1,6
      LM(M)=ID(NI,M)
      LM(M+12)=C
      LM(M+18)=C
  170 LM(M+6)=ID(NJ,M)
      NS=12
      ND=12
C
C     TRANSFORM TO MASTER DEGREES OF FREEDOM
C
      CALL SLAVE (X,Y,Z,ID,NUMNP,NI,NJ)
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITET (PBAND,NDIF)
C
C     SET NONLINEAR ELEMENT INDICATOR AND STORE NONLINEAR PARAMETERS
C
      NN=NUPEL+NEL
      IF (NIND.LE.C) GO TO 200
      INO(NN)=3
      DO 190 I=1,12
  190 ENPAR(I)=ENPAC(NIND,I)
      MTYPE=3
      WRITE (3) MTYPE,LM,ND,NS,ASA,SA,S,ENPAR,TT
      GO TO 250
  200 INO(NN)=-3
C
C     CHECK FOR LAST ELEMENT
C
  250 IF (NCBEAM-NEL) 66,500,60
  500 RETURN
C
 1000 FORMAT(I5,F10.0)
 1001 FORMAT(I5,3F10.0)
 1002 FORMAT(I5,6F10.0)
 1005 FORMAT(I5,6F10.0/F15.0,5F10.0)
 1006 FORMAT (4F10.0)
 1007 FORMAT (I5,4F10.0/8F10.0)
 2000 FORMAT (26H1RADII OF CURVED BEAMS....  //
     .        27H RADIUS        RADIUS OF     /
     .        27H NUMBER        CURVATURE     /)
 2001 FORMAT (////24H MATERIAL PROPERTIES....  //
     .        54H MATERIAL   YOUNG S     POISSON S             MASS     /
     .        54H NUMBER     MODULUS       RATIO             DENSITY    /)
 2002 FORMAT(1H ,I5,3X,F12.0,F14.5,F14.5)
 2003 FORMAT (38H1CURVED BEAM GEOMETRIC PROPERTIES.....  //
     1   48H ELEMENT       AREA          AREA          AREA
     2  60H INERTIA     INERTIA     INERTIA
     3 / 48H  TYPE             X             Y             Z
     4                  30H     X             Y             Z           /)
 2004 FORMAT(1H ,I5,2X,6F12.3)
 2005 FORMAT(39H1.......THREE DIM. CURVED BEAM ELEMENTS///
     .  36H  NUMBER OF CURVED BEAMS              =,I5//
     .  36H  NUMBER OF SECTIONAL PROPERTY SETS=,I5//
     .  36H  NUMBER OF FIXED END FORCE SETS    =,I5//
     .  36H  NUMBER OF MATERIALS               =,I5//
     .  36H  NUMBER OF RADII                   =,I5//
     .  36H  NUMBER OF N/L PARAMETER SETS      =,I5)
 2006 FORMAT (///30H ELEMENT LOAD MULTIPLIERS...../12X,1HA,14X,1HB,14X,
     .  1HC,14X,1HD/6H X-DIR 4E15.6/6H Y-DIR 4E15.6/6H Z DIR 4E15.6/  )
 2010 FORMAT (1H1
     1  30X40H FIXED END FORCES IN LOCAL COORDINATES
     2//53H TYPE    NODE          FORCE X      FORCE Y      FORCE Z
     3                      35H MOMENT X     MOMENT Y     MOMENT Z        ///)
 2011 FORMAT(1H ,I3,6X,1HI,3X,6F12.3/1H ,9X,1HJ,3X,6F12.3/)
 2013 FORMAT(1H0/
     1  60H SECTION PROPERTIES OTHER THAN SHEAR AREAS MAY NOT BE SPECIF
     2  34HIED AS ZERO. EXECUTION TERMINATED.)
 2014 FORMAT(1H0/
     1  60H RADIUS OF CURVATURE MAY NOT BE SPECIFIED
     2  34H     AS ZERO. EXECUTION TERMINATED.)
 2015 FORMAT (I5,F20.5)
 2020 FORMAT (1H1/30X 40HCURVED BEAM ELEMENT NONLINEAR PARAMETERS  //
     .  8H NL PAR  4X 8HAXIAL     4X 8HMOMENT   4X 8HMOMENT   4X
     .  8HTORSION  20X 26HYIELD FUNCTION CONSTANTS /
     .  8H     NO    6X 2HPO 9X 4HMO/Y 8X 4HMO/Z 9X 2HTO 8X 2HAO 6X 2HA1
     .  6X 2HA2 6X 2HA3 6X 2HB0 6X 2HB1 6X 2HB2 6X 2HB3  /)
 2021 FORMAT (I5,3X,4E12.3,8F8.3)
 3000 FORMAT (10I5,2I6,I6,I5)
 4000 FORMAT (30H1CURVED BEAM ELEMENT DATA.....  //
     . 5HBEAM 5X 5HNODES 5X 5H MATL 5H GEOM 5X 10HELEM LOADS 4X    10X
     . 12H  END CODES    5X 6HRADIUS 3X 5H NIND /
     .                        5H  NO      4X 1HI 4X 1HJ 4X 1HK 5H  NO  5H  NO
     .  4X 1HA 4X 1HB 4X 1HC 4X 1HD 9X 1HI 9X 1HJ    8X 2HNO 8H       NO /)
 4001 FORMAT (10I5,2I10,I10,I8)
 4006 FORMAT(1H ,31HNODAL POINT NUMBERS FOR ELEMENT,I5,36HARE IDENTICAL.
     1 EXECUTION TERMINATED.)
```

```
C
      END

      SUBROUTINE NEWCBM (E,G,RO,COPROP,SFT,RAD,NUMFIX,NUMETP)
C
      DIMENSION E(1),G(1),RO(1),COPROP(NUMETP,1),SFT(NUMFIX,1),RAD(1)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     .  SF(12,4),ENPAR(24),S(12,12),F(48),LMS(12),EC(12),TI(3,3),TJ(3,3)
      COMMON/JUNK/ LC(4),JK(6),MELTYP,DL,MATTYP,BETA,B,DP,D,RA,AX,
     .                FMUL(3,4),ILC(4),LS(4),R(12),C(12),EL(3,4),V(12,4)
C
C     FORM NEW CURVED BEAM STIFFNESS
C
      DO 5 I=1,144
    5 S(I)=0.
      AX=COPROP(MELTYP,1)
      AY=COPROP(MELTYP,2)
      AZ=COPROP(MELTYP,3)
      AAX=COPROP(MELTYP,4)
      AAY=COPROP(MELTYP,5)
      AAZ=COPROP(MELTYP,6)
C
      RA2=RA*RA
      RA3=RA2*RA
      EIY=E(MATTYP)*AAY
      EIZ=E(MATTYP)*AAZ
      PHIZ=EIZ/(AX*RA2)
      PHIY=EIY/(G(MATTYP)*AAX)
C
      IF (AY.LE.0.) GO TO 20
      ZETAZ=EIZ/(AY*G(MATTYP)*RA2)
      GO TO 21
   20 ZETAZ=0.
   21 IF (AZ.LE.0.) GO TO 30
      ZETAY=EIY/(AZ*G(MATTYP)*RA2)
      GO TO 31
   30 ZETAY=0.
   31 CONTINUE
      COMMY=RA3/EIY
      COMMZ=RA3/EIZ
C
C     FIXED END FORCES IN LOCAL COORDINATES
C
      DO 73 N=1,4
      M=LC(N)
      IF (M.GT.0)  GO TO 71
      DO 70 I=1,12
   70 SF(I,N)=0.
      GO TO 73
   71 DO 72 I=1,12
   72 SF(I,N)=SFT(M,I)
   73 CONTINUE
C
C     FORM ELEMENT STIFFNESS IN LOCAL COORDINATES
C
      AA=BETA*0.5-0.5*B*DP
      CC=BETA*0.5+0.5*B*DP
      EE=0.5*B*B
C
      F11=COMMZ*(BETA+CC-2.0*B+PHIZ*CC+ZETAZ*AA)
      F12=COMMZ*(D-EE-PHIZ*EE+ZETAZ*EE)
      F16=(COMMZ/RA)*(B-BETA)
      F22=COMMZ*(AA+PHIZ*AA+ZETAZ*CC)
      F26=(COMMZ/RA)*(-D)
C
      F33=COMMY*(AA+PHIY*(BETA+CC-2.0*B)+ZETAY*BETA)
      F34=(COMMY/RA)*(AA-PHIY*(B-CC))
      F35=(COMMY/RA)*(EE+PHIY*(D-EE))
      F44=(COMMY/RA2)*(AA+PHIY*CC)
      F45=(COMMY/RA2)*(EE-PHIY*EE)
      F55=(COMMY/RA2)*(CC+PHIY*AA)
      F66=(COMMZ/RA2)*BETA
C
      U=F11*F22*F66+2.0*F12*F16*F26-F22*F16*F16-F11*F26*F26-F66*F12*F12
      W=F33*F44*F55+2.0*F34*F45*F35-F44*F35*F35-F33*F45*F45-F55*F34*F34
C
      S(1,1)= (F22*F66-F26*F26)/U
      S(1,2)=-(F12*F66-F16*F26)/U
      S(1,6)= (F12*F26-F22*F16)/U
      S(2,2)= (F11*F66-F16*F16)/U
      S(2,6)=-(F11*F26-F12*F16)/U
      S(6,6)= (F11*F22-F12*F12)/U
C
      S(3,3)= (F44*F55-F45*F45)/W
      S(3,4)=-(F34*F55-F35*F45)/W
      S(3,5)= (F34*F45-F44*F35)/W
      S(4,4)= (F33*F55-F35*F35)/W
      S(4,5)=-(F33*F45-F35*F34)/W
      S(5,5)= (F33*F44-F34*F34)/W
C
      S(1, 7)= S(1,1)*DP+S(1,2)*B
      S(1, 8)= S(1,1)*B-S(1,2)*DP
      S(1,12)= S(1,1)*RA*D+S(1,2)*RA*B-S(1,6)
      S(2, 7)= S(1,2)*DP+S(2,2)*B
      S(2, 8)= S(1,2)*B-S(2,2)*DP
      S(2,12)= S(1,2)*RA*D+S(2,2)*RA*B-S(2,6)
      S(3, 9)= S(3,3)
      S(3,10)= S(3,3)*RA*D-S(3,4)*DP+S(3,5)*B
      S(3,11)= S(3,3)*RA*B-S(3,4)*B-S(3,5)*DP
      S(4, 9)= S(3,4)
      S(4,10)= S(3,4)*RA*D-S(4,4)*DP+S(4,5)*B
      S(4,11)= S(3,4)*RA*B-S(4,4)*B-S(4,5)*DP
      S(5, 9)= S(3,5)
      S(5,10)= S(3,5)*RA*D-S(4,5)*DP+S(5,5)*B
      S(5,11)= S(3,5)*RA*B-S(4,5)*B-S(5,5)*DP
      S(6, 7)= S(1,6)*DP+S(2,6)*B
      S(6, 8)= S(1,6)*B-S(2,6)*DP
      S(6,12)= S(1,6)*RA*D+S(2,6)*RA*B-S(6,6)
C
      S( 7, 7)= S(1,1)
      S( 7, 8)=-S(1,2)
      S( 7,12)= S(1,6)
      S( 8, 8)= S(2,2)
      S( 8,12)=-S(2,6)
      S( 9, 9)= S(3,3)
      S( 9,10)= S(3,4)
      S( 9,11)=-S(3,5)
      S(10,10)= S(4,4)
      S(10,11)=-S(4,5)
      S(11,11)= S(5,5)
      S(12,12)= S(6,6)
```

```
C
      DO 106 I=2,12
      K=I-1
      DO 106 J=1,K
  106 S(I,J)=S(J,I)
C
C     ELEMENT LOCAL FORCES DUE TO GRAVITIES IN 3 DIRECTIONS
C
      DO 107 I=1,6C
  107 EL(I)=0.
      ROM=RC(MATTYP)*AX
      DO 110 J=1,4
      DO 110 I=1,3
      XX=0.
      DO 109 K=1,3
  109 XX=XX+RCM*TI(I,K)*EMUL(K,J)
  110 EL(I,J)=XX
C
      V11=COMMZ*((2.0*D-0.75*B*B-BETA*B+0.5*BETA*B*DP+0.25*BETA*BETA)
     .              +PHIZ*(0.5*BETA*B*DP+0.25*BETA*BETA-0.25*B*B))
      V12=COMMZ*((3.0*B-BETA+DP-C.75*B*DP+0.5*BETA*DP*DP-0.75*BETA)
     .              -PHIZ*(0.5*BETA*B*R+0.25*R*DP))
      V21=COMMZ*((0.25*BETA-0.75*B*DP+0.5*BETA*DP*DP)
     .              -PHIZ*(0.5*BETA+0.25*R*DP-0.5*BETA*DP*DP))
      V22=COMMZ*((C.25*BETA*BETA-0.5*BETA*R*DP+0.5*B*B-0.25*DP*DP+DP
     .      -C.75)+PHIZ*(0.25*BETA*BETA-0.5*BETA*B*DP+0.25*B*B))
      V3 =COMMY*((1.0-DP-0.5*B*B)+PHIY*(0.5*BETA*BETA-BETA*B+0.5*B*B))
      V4 =(COMMY/RA)*((1.0-DP-0.5*B*B)-PHIY*(DP+BETA*B-0.5*B*B-1.0))
      V5 =(COMMY/RA)*((B-0.5*BETA-0.5*B*DP)
     .                   +PHIY*(B-0.5*BETA-BETA*DP+0.5*B*DP))
      V61=(COMMZ/RA)*(BETA*B-2.0+2.0*DP)
      V62=(COMMZ/RA)*(BETA+BETA*DP-2.0*B)
C
      R71=-RA*BETA*CP
      R72= RA*BETA*B
      R81=-R72
      R82= R71
      R9 =-RA*BETA
      R10= RA2*(BETA B)
      R11=-RA2*C
      R121=RA2*(B BETA*DP)
      R122=RA2*(BETA*B+DP-1.0)
C
      DO 120 J=1,4
      V(1,J)=EL(1,J)*V11+EL(2,J)*V12
      V(2,J)=EL(1,J)*V21+EL(2,J)*V22
      V(3,J)=EL(3,J)*V3
      V(4,J)=EL(3,J)*V4
      V(5,J)=EL(3,J)*V5
      V(6,J)=EL(1,J)*V61+EL(2,J)*V62
C
      SF( 7,J)=SF( 7,J)+EL(1,J)*R71+EL(2,J)*R72
      SF( 8,J)=SF( 8,J)+EL(1,J)*R81+EL(2,J)*R82
      SF( 9,J)=SF( 9,J)+EL(3,J)*R9
      SF(10,J)=SF(10,J)+EL(3,J)*R10
      SF(11,J)=SF(11,J)+EL(3,J)*R11
      SF(12,J)=SF(12,J)+EL(1,J)*R121+EL(2,J)*R122
  120 CONTINUE
C
      DO 124 I=1,12
      DO 124 L=1,4
      XX=0.
      DO 123 K=1,6
  123 XX=XX-S(I,K)*V(K,L)
  124 SF(I,L)=SF(I,L)+XX
C
C     MODIFY STIFFNESS AND FIXED END FORCES FOR ZERO END CONDITIONS
C
      IF ((JK(1)+JK(2)).EQ.0) GO TO 145
      DO 14C K=1,2
      KK=JK(K)
      KD=100000
      I1=6*(K-1)+1
      I2=I1+5
      DO 14C I=I1,I2
      IF (KK.LT.KD)  GO TO 140
      SII=S(I,I)
      DO 125 N=1,12
  125 R(N)=S(I,N)
      DO 13C M=1,12
      C(M)=S(M,I)/SII
      DO 130 N=1,12
  130 S(M,N)=S(M,N)-C(M)*R(N)
      DO 135 N=1,4
      SFI=SF(I,N)
      DO 135 M=1,12
  135 SF(M,N)=SF(M,N)-C(M)*SFI
  136 KK=KK-KD
  140 KD=KD/10
  145 CONTINUE
C
      RETURN
      END

      SUBROUTINE TRANSFM (RO,COPROP,NUMETP)
C
      DIMENSION RO(1),COPROP(NUMETP,1)
      COMMON/EM/LM(24),NU,NS,ASA(24,24),PF(24,4),XM(24),SA(12,24),
     . SF(12,4),FNPAR(24),S(12,12),F(48),LMS(12),EC(12),TI(3,3),TJ(3,3)
      COMMON/JUNK/ LC(4),JK(6),MELTYP,OL,MATTYP,BETA,B,DP,D,RA,AX,
     .              EMUL(3,4),ILC(4),LS(4),R(12),C(12),EL(3,4)
C
C     PERFORM LOCAL TO GLOBAL TRANSFORMATION
C
      DO 31 I=1,288
   31 SA(I)=0.
      DO 15C LA=1,1C,3
      LB=LA+2
      DO 15C MA=1,1C,3
      MB=MA-1
      DO 15C I=LA,LB
      DO 15C JM=1,3
      J=JM+MB
      XX=0.
      IF (J.GT.6) GC TO 152
      DO 151 K=1,3
  151 XX=XX+S(I,K+MB)*TI(K,JM)
      GO TO 150
  152 DO 153 K=1,3
  153 XX=XX+S(I,K+MB)*TJ(K,JM)
  150 SA(I,J)=XX
```

```
C
      DO 32 I=1,576
   32 ASA(I)=0.
C
      DO 16C LA=1,1C,3
      LB=LA-1
      DO 16C MA=1,1C,3
      MB=MA+2
      DO 16C IL=1,3
      I=IL+LB
      DO 160 J=MA,MB
      XX=0.
      IF (I.GT.6) GC TO 162
      DO 161 K=1,3
  161 XX=XX+TI(K,IL)*SA(K+LB,J)
      GO TO 160
  162 DO 163 K=1,3
  163 XX=XX+TJ(K,IL)*SA(K+LB,J)
  160 ASA(I,J)=XX
C
      DO 17C LA=1,1C,3
      LB=LA-1
      DO 17C IL=1,3
      I=IL+LB
      DO 170 N=1,4
      XX=0.
      IF (I.GT.6) GC TO 172
      DO 171 K=1,3
  171 XX=XX-TI(K,IL)*SF(K+LB,N)
      GO TC 170
  172 DO 173 K=1,3
  173 XX=XX-TJ(K,IL)*SF(K+LB,N)
  170 FF(I,N)=XX
C
C     FORM MASS MATRIX
C
      XL  =RA*BETA
      XXM =RO(MATTYP)*AX*XL/2.0
      XXMX=XXM*(COPROP(MATTYP,4)/AX)
      XXMY=XXM*(XL**2/210.+COPROP(MATTYP,5)/(5.0*AX))
      XXMZ=XXM*(XL**2/210.+COPROP(MATTYP,6)/(5.0*AX))
      DO 18C M=1,3
      XM(M  )=XXM
      XM(M+6)=XXM
      XM(M+3)=XXMX*TI(1,M)**2+XXMY*TI(2,M)**2+XXMZ*TI(3,M)**2
  180 XM(M+9)=XXMX*TJ(1,M)**2+XXMY*TJ(2,M)**2+XXMZ*TJ(3,M)**2
      RETURN
      END
```

```
      SUBROUTINE BOUND
C
      COMMON A(1)
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEC,NEQG,MBAND,
     .                   NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/JUNK/ STR(4),MM,L,K,NTAG,NDYN,SIG(12),EXPA(184)
C
      IF (NPAR(1).EQ.0) GO TO 500
      N7=N6+NPAR(3)*6
      N8=N7+NPAR(4)*12
      IF (N8.GT.MTOT) CALL ERROR (1,N8-MTOT,7HBOUND   )
      CALL BIND (NPAR(2),NPAR(3),NPAR(4),A(N1),A(N2),A(N3),A(N4),A(N5),
     .             A(N6),A(N7),NUMNP,MBAND,NUMEL)
      RETURN
C
  500 IF (NTAG.EQ.C) WRITE (6,2002)
      WRITE(6,3C02) MM,L,(SIG(I),I=1,6 )
      NTAG=1
      RETURN
C
 2002 FORMAT (/23HC ....BOUNDARY FORCES AND MOMENTS  //
     .  10HO BD. LOAD 5X 5HAXIAL 2(7X 5HSHEAR) 5X 7HTORSION
     .  2(5X 7HBENDING) /10H   NO    NO  8X 2HR1 10X 2HR2 10X 2HR3 10X
     .  2HM1 10X 2HM2 10X 2HM3 )
 3002 FORMAT (I5,I4,1PE11.3,5E12.3)
C
      END
```

```
      SUBROUTINE BIND (NBOUND,NSTIF,NUMNPAR,ID,X,Y,Z,INC,STIF,ENPROP,
     .                 NUMNP,MBAND,NUMEL)
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),IND(1),STIF(NSTIF,1),
     .          ENPROP(NUMNPAR,1),ENPAR(24),T(3,3)
      COMMON/EM/LM(24),ND,NS,ASA(24,24),P(24,4),XM(24),SA(12,24),
     .          SF(12,4),SY(6),EF(6),UE(6),PAR(6),S(12,12),F(48),TTT(72)
      COMMON/JUNK/ DX,DY,DZ,DD,AX,AY,AZ,AA,BX,BY,BZ,BB,SPRING(6),NP
      EQUIVALENCE (ENPAR,SY),(T,TTT)
C
C     FORM ELEMENT STIFFNESS OF BOUNDARY SPRING ELEMENTS
C
      DO 5 I=1,1346
    5 LM(I)=0.
      ND=6
      NS=6
      NE=0
      WRITE(6,2COC) NBOUND,NSTIF,NUMNPAR
C
C     READ AND PRINT SPRING STIFNESS SETS
C
      READ (5,1010) (N,(STIF(N,J),J=1,6),N=1,NSTIF)
      WRITE(6,2C01)
      WRITE(6,2C1C) (N,(STIF(N,J),J=1,6),N=1,NSTIF)
C
C     READ AND PRINT NONLINEAR ELEMENT PARAMETERS
C
      IF (NUMNPAR.EQ.0) GO TO 9
      READ (5,1C20) (N,(ENPROP(N,J),J=1,12),N=1,NUMNPAR)
      WRITE(6,2C02)
```

```
      WRITE(6,2010) (N,(ENPROP(N,J),J=1,6 I,N=1,NUMNPAR)
      WRITE(6,2C03)
      WRITE(6,2C10) (N,(ENPROP(N,J),J=7,12),N=1,NUMNPAR)
    9 CONTINUE
C
C     READ AND PRINT ELEMENT DATA---DATA NEED TO BE SUPPLIED FOR EACH
C
      WRITE(6,2C04)
   10 NE=NE+1
      READ (5,1030) INE,NI,NJ,NK,NS,NIND
      IF (INE.NE.NE) CALL ERROR (4,INE,7HBOUND   )
      IF (NIND.GT.NUMNPAR) CALL ERROR (3,INE,7HBOUND   )
      WRITE(6,2C4C) NE,NI,NJ,NK,NS,NIND
C
C     FORM LOCAL TC GLOBAL TRANSFORMATION
C
      DX=X(NJ)-X(NI)
      DY=Y(NJ)-Y(NI)
      DZ=Z(NJ) Z(NI)
      DD=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF (DD.LE.0.) CALL ERROR (5,INE,7HBOUND   )
      AX=X(NK)-X(NI)
      AY=Y(NK)-Y(NI)
      AZ=Z(NK)-Z(NI)
      BX=DY*AZ-DZ*AY
      BY=DZ*AX-DX*AZ
      BZ=DX*AY DY*AX
      BB=SQRT(BX*BX+BY*BY+BZ*BZ)
      IF (BB.LE.0.0) CALL ERROR (6,NE,7HBOUND   )
C
      T(3,1)=BX/BB
      T(3,2)=BY/BB
      T(3,3)=BZ/BB
      T(1,1)=DX/DD
      T(1,2)=DY/DD
      T(1,3)=DZ/DD
      T(2,1)=T(3,2)*T(1,3)-T(3,3)*T(1,2)
      T(2,2)=T(3,3)*T(1,1)-T(3,1)*T(1,3)
      T(2,3)=T(3,1)*T(1,2)-T(3,2)*T(1,1)
C
C     FORM BOUNDARY STIFFNESSES IN LOCAL COORDINATE SYSTEM
C
      DO 100 I=1,144
  100 S(I)=C.
      DO 110 I=1,6
      SPRING(I)=STIF(NS,I)
      S(I,I)=SPRING(I)
  110 CONTINUE
C
C     FORM LOCAL TC GLOBAL STRESS ARRAY
C
      DO 12C I=1,288
  120 SA(I)=0.
      DO 15C LA=1,6,3
      LB=LA+2
      DO 15C MA=1,6,3
      MB=MA-1
      DO 15C I=LA,LB
      DO 15C JM=1,3
      J=JM+MB
      XX=0.
      DO 151 K=1,3
      YY=S(I,K+MB)
      IF (YY.EQ.C.) GO TO 151
      XX=XX+YY*T(K,JM)
  151 CONTINUE
  150 SA(I,J)=XX
C
C     FORM ELEMENT STIFFNESSES IN GLOBAL COORDINATE SYSTEM
C
      DO 155 I=1,576
  155 ASA(I)=0.
      DO 16C LA=1,6,3
      LB=LA-1
      DO 160 MA=1,6,3
      MB=MA+2
      DO 16C IL=1,3
      I=IL+LB
      DO 16C J=MA,MB
      XX=0.
      DO 161 K=1,3
      YY=SA(K+LB,J)
      IF (YY.EQ.0.) GO TO 161
      XX=XX+T(K,IL)*YY
  161 CONTINUE
  160 ASA(I,J)=XX
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 17C I=1,6
      LM(I)=ID(NI,I)
      LM(I+6)=C
      LM(I+12)=0
  170 LM(I+18)=C
C
C     WRITE ELEMENT INFORMATION CN TAPE
C
      CALL WRITET (MBAND,NDIF)
C
C     SET NONLINEAR ELEMENT INDICATORS AND STORE NONLINEAR PARAMETERS
C
      NN=NUMEL+NE
      IF (NIND.EQ.C) GO TO 200
      IND(NN)=4
      DO 18C I=1,6
      II=I+6
      SY(I)=ENPROP(NIND,I)
      EP(I)=ENPROP(NIND,II)
      UE(I)=SY(I)/SPRING(I)
  180 CONTINUE
      MTYPE=4
      WRITE (3) MTYPE,LM,ND,NS,ASA,SA,S,ENPAR,TTT
      GO TC 250
  200 IND(NN)=-4
C
  250 IF (NE.LT.NBCLND) GO TO 10
      RETURN
C
 1010 FORMAT (I5,6F10.0)
 1020 FORMAT (I5,6F10.0/5X,6F10.0)
 1030 FORMAT (6I5)
C
 2000 FORMAT (24H1......BOUNDARY ELEMENTS  ///
     ,  28H NUMBER CF ELEMENTS         =,I5//
```

```
      .  28H NUMBER CF STIFFNESS SETS   =,I5//
      ,  28H NUMBER CF N/L PAR. SETS    =,I5)
 2001 FORMAT (4H1SET 20X 31HSTIFFNESSES OF BOUNDARY SPRINGS /
      .  4H NC   4X 2HR1 10X 2HR2 10X 2HR3 10X 2HM1 10X 2HM2 10X 2HM3 /)
 2002 FORMAT (4H1SET 20X 35HYIELDING FORCES OF BOUNDARY SPRINGS /
      .  4H NC   4X 2HP1 10X 2HR2 10X 2HR3 10X 2HM1 10X 2HM2 10X 2HM3 /)
 2003 FORMAT (4H1SET 20X 39HPLASTIC STIFFNESSES OF BOUNDARY SPRINGS /
      .  4H NC   4X 2HR1 10X 2HR2 10X 2HR3 10X 2HM1 10X 2HM2 10X 2HM3 /)
 2004 FORMAT (5H1ELE. 5X 5HNODES 5X 5H MATL 5X 5H NINO /
      ,  5H  NO  4X 1HI 4X 2HJ 4X 1HK 5H  NO  5X 5H  NO  /)
 2010 FORMAT (I4,6E12.3)
 2040 FORMAT (5I5,11C)
C
      END
```

```
      SUBROUTINE EXPJB
C
      COMMON/ELPAR/ NPAR(14),NUMNP,NELTYP,NUMEL,NUMNEL,NEQ,NEQG,MBAND,
     .               NEQB,NBLOCK,MTOT,N1,N2,N3,N4,N5,N6
      COMMON/JUNK/ STR(4),MM,L,K,NTAG,NDYN,SIG(12),EXPA(184)
      COMMON A(1)
C
      IF (NPAR(1).EQ.0) GO TO 500
      N7=N6+NPAR(3)*5
      N8=N7+NPAR(3)
      N9=N8+NPAR(3)*10
      IF (N9.GT.MTOT) CALL ERROR (1,N7-MTOT,7HEXPJB   )
      CALL XPJB (NPAR(2),NPAR(3),A(N1),A(N2),A(N3),A(N4),A(N5),A(N6),
     .           A(N7),A(N8),NUMNP,MBAND,NUMEL)
      RETURN
C
  500 IF (NTAG.EQ.0) WRITE(6,2002)
      WRITE(6,3002) MM,L,(SIG(I),I=1,12)
      NTAG=1
      RETURN
C
 2002 FORMAT (/30HC ....JOINT FORCES AND MOMENTS  //
     .  10H0 JT. LCAD,5X,5HAXIAL,2(7X,5HSHEAR),7X,5HAXIAL,5X,7HBENDING,
     .  5X,7H  SHEAR/10H  NO.  NC, ,8X,2HF1,10X,2HF2,10X,2HF3,
     .  10X,2HR4,10X,2HM5,10X,2HR6)
 3002 FORMAT (I5,I4,1PE11.3,5E12.3/8X,6E12.3/)
C
      END
```

```
      SUBROUTINE XPJP (NEXPJ,NUMNPAR,ID,X,Y,Z,INO,ENPROP,NT,XT,NUMNP,
     .                 MBAND,NUMEL)
C
      DIMENSION ID(NUMNP,1),X(1),Y(1),Z(1),INO(1),ENPROP(NUMNPAR,1),
     .          NT(1),XT(NUMNPAR,1),ENPAR(24),AT(6,6),T(3,3)
      COMMON/EM/ LM(24),ND,NS,ASA(24,24),P(24,4),XM(24),SA(12,24),
     .          SF(12,4),COF,SL,GAP,STIE,SYTIE,MTIE,ATIE(8),TFACE,
     .          D,ULTIF,SJ,XD(6),S(12,12),F(48),TTT(72)
      COMMON/JUNK/ SC,TSJ,CSU,A(6,6)
      EQUIVALENCE (ENPAR,COF),(AT,TTT),(T,TTT(37))
C
C     FORM 3-D STIFFNESSES FOR EXPANSION JOINT ELEMENTS
C
      DO 5 I=1,1346
    5 LM(I)=0.
      ND=12
      NS=12
      NE=0
      WRITE(6,2000) NEXPJ,NUMNPAR
C
C     READ AND PRINT NONLINEAR ELEMENT PARAMETERS
C
      IF (NUMNPAR.EQ.0) GO TO 9
      READ (5,1010) (N,(ENPROP(N,J),J=1,5),NT(N),(XT(N,J),J=1,6),
     .               N=1,NUMNPAR)
      WRITE(6,2001)
      WRITE(6,2002) (N,(ENPROP(N,J),J=1,5),N=1,NUMNPAR)
      WRITE(6,2003)
      WRITE(6,2004) (N,NT(N),(XT(N,J),J=1,6),N=1,NUMNPAR)
```

```
    9 CONTINUE
C
C     READ AND PRINT ELEMENT DATA---DATA NEED TO BE SUPPLIED FOR EACH
C
      WRITE(6,2010)
   10 NE=NE+1
      READ (5,1030) INE,NI,NJ,NK,NL,KD,JS,NIND,D,SQ,TRACE
      IF (NIND.GT.NUMPAR) CALL ERROR (3,INE,7HEXPLB  )
      IF (INE.NE.NE) CALL ERROR (4,INE,7HEXPJB  )
      IF (TRACE.EQ.0.) TRACE=1.0E+10
      WRITE(6,2020) NE,NI,NJ,NK,NL,KD,JS,NIND,D,SQ,TRACE
C
C     FORM TO LOCAL TRANSFORMATION
C
      IF (SQ.EQ.0.) GO TO 20
      SQ=SQ*3.1415927/180.
      TSQ=TAN(SQ)
      CSQ=COS(SQ)
      GO TO 21
   20 SQ=0.
      TSQ=0.0
      CSQ=1.0
   21 DO 25 I=1,6
      DO 25 J=1,6
   25 A(I,J)=0.
C
      A(1,1)=1.
      A(1,6)=0.5*D
      A(2,2)=1.
      A(3,3)=1.
      A(3,4)=-A(1,6)
      A(3,5)=0.5*D*TSQ
      A(4,1)=1.
      A(4,6)=-A(1,6)
      A(5,5)=1.0/CSQ
      A(6,3)=1.0
      A(6,4)=A(1,6)
      A(6,5)=-A(3,5)
C
C     FORM LOCAL TO GLOBAL TRANSFORMATION
C
      DX=X(NL)-X(NI)
      DY=Y(NL)-Y(NI)
      DZ=Z(NL)-Z(NI)
      DL=SQRT(DX*DX+DY*DY+DZ*DZ)
      IF (DL) 55,55,60
   55 CALL ERROR (5,NE,7HEXPJB  )
C
   60 AX=X(NK)-X(NI)
      AY=Y(NK)-Y(NI)
      AZ=Z(NK)-Z(NI)
      BX=AY*DZ-AZ*DY
      BY=AZ*DX-AX*DZ
      BZ=AX*DY-AY*DX
      BB=SQRT(BX*BX+BY*BY+BZ*BZ)
      IF (BB) 65,65,70
   65 CALL ERROR (6,NE,7HEXPJB  )
C
   70 T(3,1)=BX/BB
      T(3,2)=BY/BB
      T(3,3)=BZ/BB
      AA=SQRT(AX*AX+AY*AY+AZ*AZ)
      IF (AA) 65,65,80
   80 T(2,1)=-AX/AA
      T(2,2)=-AY/AA
      T(2,3)=-AZ/AA
      T(1,1)=T(2,2)*T(3,3)-T(2,3)*T(3,2)
      T(1,2)=T(2,3)*T(3,1)-T(2,1)*T(3,3)
      T(1,3)=T(2,1)*T(3,2)-T(2,2)*T(3,1)
C
C     FORM JOINT TO GLOBAL TRANSFORMATION
C
      DO 90 I=1,6,3
   90 AT(I)=0.
      DO 100 LA=1,6,3
      LB=LA+2
      DO 100 MA=1,6,3
      MB=MA-1
      DO 100 I=LA,LB
      DO 100 JM=1,3
      J=JM+MB
      XX=0.
      DO 95 K=1,3
      XX=XX+A(I,K+MB)*T(K,JM)
   95 CONTINUE
  100 AT(I,J)=XX
C
C     FORM JOINT STIFFNESS IN JOINT COORDINATE SYSTEM
C
      DO 101 I=1,144
  101 S(I)=0.
      DO 110 I=1,6
      IF (KD(I).EQ.0) GO TO 110
      S(I,I)=TRACE
      S(I,I+6)=-TRACE
      S(I+6,I)=-TRACE
      S(I+6,I+6)=TRACE
  110 CONTINUE
C
C     FORM JOINT-GLOBAL STRESS ARRAY
C
      DO 120 I=1,288
  120 SA(I)=0.
      DO 130 LA=1,12,6
      LB=LA+5
      DO 130 MA=1,12,6
      MB=MA-1
      DO 130 I=LA,LB
      DO 130 JM=1,6
      J=JM+MB
      XX=0.
      DO 125 K=1,6
      YY=S(I,K+MB)
      IF (YY.EQ.0.) GO TO 125
      XX=XX+YY*AT(K,JM)
  125 CONTINUE
  130 SA(I,J)=XX
C
C     FORM JOINT STIFFNESS IN GLOBAL COORDINATE SYSTEM
C
      DO 151 I=1,576
  151 ASA(I)=0.
      DO 160 LA=1,12,6
      LB=LA-1
```

```
      DO 160 MA=1,12,6
      MB=MA+5
      DO 160 IL=1,6
      I=IL+LB
      DO 160 J=MA,MB
      XX=0.
      DO 155 K=1,6
      XX=XX+AT(K,IL)*SA(K+LB,J)
  155 CONTINUE
  160 ASA(I,J)=XX
C
C     FORM ELEMENT LOCATION MATRIX
C
      DO 170 I=1,6
      LM(I)=ID(NI,I)
      LM(I+6)=ID(NJ,I)
      LM(I+12)=C
  170 LM(I+18)=C
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      CALL WRITFT (MBAND,NDIF)
C
C     SET NONLINEAR ELEMENT INDICATOR AND STORE NONLINEAR PARAMETERS
C
      NN=NUMEL+NE
      IF (NIND.EQ.0) GO TO 200
      IND(NN)=5
      DO 180 I=1,5
  180 ENPAR(I)=ENPROP(NIND,I)
      NTIE=NT(NIND)
      DO 190 I=1,6
  190 XTIE(I)=XT(NIND,I)
      UETIE=SYTIE/STIE
      SJ=JS
      MTYPE=5
      WRITE (3) MTYPE,LM,ND,NS,ASA,SA,S,ENPAR,TTT
      GO TO 250
  200 IND(NN)=-5
C
  250 IF (NE.LT.NEXPJ) GO TO 10
      RETURN
C
 1010 FORMAT (I5,5F10.0,I5/ 6F10.0)
 1030 FORMAT (5I5,6I1,I4,I5,2F10.0,F20.0)
 2000 FORMAT (31H1.....EXPANSION JOINT ELEMENTS    ///
     .  29H NUMBER OF ELEMENTS          =  I5//
     .  29H NUMBER OF N/L PAR. SETS     =  I5)
 2001 FORMAT (53H1NONLINEAR PARAMETERS OF EXPANSION JOINT ELEMENTS....//
     1  53H NPAR  FRICTION  SEAT     TIE      TIE          TIE YIELD     /
     2  53H  NO.  COEFF.    GAP      GAP      STIFF.         FORCE       )
 2002 FORMAT (I5,3F8.3,2E12.3)
 2003 FORMAT (////20H JOINT TIE BARS.....  //
     1  56H NPAR  NUMBER      TIE POSITIONS RELATIVE TO JOINT CENTER     /
     2  59H  NO.  OF TIES   1        2        3        4        5        6  )
 2004 FORMAT (I5,I5,3X,6F8.3)
 2010 FORMAT (////34H EXPANSION JOINT ELEMENT DATA.....  //
     .  5H ELEM 8X 4HNODE 11X 6H JOINT 3X 5HJOINT 5H  N/L
     1  8H   JOINT 8H    SKEW 8H   JOINT /
     2  5H  NO. 4X 1HI 4X 1HJ 4X 1HK 4X 1HL 3X 5H CODE 5X 4HSIGN 5H  IND
     3  8H   WIDTH 9H    ANGLE 2X 9HSTIFFNESS  /)
 2020 FORMAT (5I5,3X,6I1,3X,2I5,2F8.3,E12.3)
      END
```

```
      SUBROUTINE PBEAM (NEL,NIND)
C
C     CHECK YIELDING OF BEAM ELEMENT AND FORM ELASTO-PLASTIC STIFFNESSES
C
      COMMON/EM/ MTYPE,LM(24),ND,NS,ASA(24,24),SA(12,24),S(12,12),PU,
     .            UMY,UMZ,TU,A0,A1,A2,A3,B0,B1,B2,B3,EPAR(12),LMS(12),
     .            EC(12),T(3,3),TTT(39),ASSA(24,24),SSA(12,24),DF(24),
     .            F(12),UP(12)
      COMMON/NEM/ EX(24),EX1(24),DEX(24),DFP(12),OP(12),FP(12),P(12),
     .            SS(12,12),U(12)
      DIMENSION FX(2),FY(2),FZ(2),FHI(2),FACT(2),
     .          PX(2),PY(2),PZ(2),PHI(2),PACT(2)
C
      DO 5 I=1,180
    5 OP(I)=0.
      DO 10 I=1,NS
   10 P(I)=F(I)
      DO 20 I=1,ND
   20 DF(I)=0.
C
      DO 60 I=1,NS
      XX=0.
      DO 50 J=1,ND
      XX=XX+SA(I,J)*DEX(J)
   50 CONTINUE
      DFP(I)=XX
   60 FP(I)=P(I)+DFP(I)
C
      FX(1)=-FP(1)/PU
      FY(1)= FP(5)/UMY
      FZ(1)= FP(6)/UMZ
      FX(2)= FP(7 )/PU
      FY(2)= FP(11)/UMY
      FZ(2)= FP(12)/UMZ
      DO 100 I=1,2
      FHI(I)=YIELD (FX(I),FY(I),FZ(I),A0,A1,A2,A3,B0,B1,B2,B3,TU)
  100 CONTINUE
C
C     FORM APPROPRIATE ELASTO-PLASTIC BEAM STIFFNESSES
C
      IF (FHI(1).GE.0.0.AND.FHI(2).LT.0.0) GO TO 300
      IF (FHI(1).LT.0.0.AND.FHI(2).GE.0.0) GO TO 400
      IF (FHI(1).GE.0.0.AND.FHI(2).GE.0.0) GO TO 500
C
C     A. BEAM REMAINING IN ELASTIC STATE
C
      DO 200 I=1,NS
  200 F(I)=FP(I)
      NIND=0
      RETURN
C
C     B. ELASTO PLASTIC BEAM WITH I-END YIELDING
C
  300 DFX=-DFP(1)/PL
      DFY= DFP(5)/UPY
      DFZ= DFP(6)/UPZ
      IF (FHI(1)) 310,310,320
  310 FACT(1)=0.
      GO TO 325
  320 FACT(1)=FACTOR (FX(1),FY(1),FZ(1),DFX,DFY,DFZ,
     .                A0,A1,A2,A3,B0,B1,B2,B3,TU    )
C
  325 EFACT=1.0-FACT(1)
      DO 330 I=1,NS
      P(I)=P(I)+EFACT*DFP(I)
  330 CONTINUE
      EFACT=FACT(1)
      CALL PTEAM (1,EFACT,NEL)
      GO TO 600
C
C     C. ELASTO-PLASTIC BEAM WITH J-END YIELDING
C
  400 DFX=DFP( 7)/PL
      DFY=DFP(11)/UPY
      DFZ=DFP(12)/UPZ
      IF (FHI(2)) 410,410,420
  410 FACT(2)=0.
      GO TO 425
  420 FACT(2)=FACTOR (FX(2),FY(2),FZ(2),DFX,DFY,DFZ,
     .                A0,A1,A2,A3,B0,B1,B2,B3,TU    )
  425 EFACT=1.0-FACT(2)
      DO 430 I=1,NS
      P(I)=P(I)+EFACT*DFP(I)
  430 CONTINUE
      EFACT=FACT(2)
      CALL PTEAM (2,EFACT,NEL)
      GO TO 600
C
C     D. ELASTO-PLASTIC BEAM WITH BOTH ENDS YIELDING
C
  500 DFX=-DFP(1)/PL
      DFY= DFP(5)/UPY
      DFZ= DFP(6)/UPZ
      IF (FHI(1)) 510,510,520
  510 FACT(1)=0.
      GO TO 525
  520 FACT(1)=FACTOR (FX(1),FY(1),FZ(1),DFX,DFY,DFZ,
     .                A0,A1,A2,A3,B0,B1,B2,B3,TU    )
C
  525 DFX=DFP( 7)/PL
      DFY=DFP(11)/UPY
      DFZ=DFP(12)/UPZ
      IF (FHI(2)) 530,530,540
  530 FACT(2)=0.
      GO TO 545
  540 FACT(2)=FACTOR (FX(2),FY(2),FZ(2),DFX,DFY,DFZ,
     .                A0,A1,A2,A3,B0,B1,B2,B3,TU    )
C
  545 EFACT=1.0-FACT(1)
      IF (FACT(1).LT.FACT(2)) EFACT=1.0-FACT(2)
      DO 550 I=1,NS
      P(I)=P(I)+EFACT*DFP(I)
  550 CONTINUE
      EFACT=FACT(1)-FACT(2)
      IF (EFACT) 570,580,560
C
  560 CALL PTEAM (1,EFACT,NEL)
      DO 565 I=1,NS
      XX=0.
      DO 561 J=1,NC
      XX=XX+SSA(I,J)*DEX(J)*EFACT
  561 CONTINUE
  565 P(I)=P(I)+XX
```

```
      EFACT=FACT(2)
      GO TO 585
C
  570 EFACT=FACT(2)-FACT(1)
      CALL PTEAM (2,EFACT,NEL)
      DO 575 I=1,NS
      XX=0.
      DO 571 J=1,NC
      XX=XX+SSA(I,J)*DEX(J)*EFACT
  571 CONTINUE
  575 P(I)=P(I)+XX
      EFACT=FACT(1)
      GO TO 585
C
  580 EFACT=FACT(1)
  585 CALL PTEAM (3,EFACT,NEL)
C
C     CHECK YIELDING OF NEW STRESS STATE
C
  600 DO 650 I=1,NS
      XX=0.
      DO 620 J=1,NC
      XX=XX+SSA(I,J)*DEX(J)*EFACT
  620 CONTINUE
  650 P(I)=P(I)+XX
C
      PX(1)=-P(1)/PL
      PY(1)= P(5)/LPY
      PZ(1)= P(6)/LPZ
      PX(2)=P( 7)/PU
      PY(2)=P(11)/LPY
      PZ(2)=P(12)/LPZ
      DO 655 I=1,2
      PHI(I)=YIELD (PX(I),PY(I),PZ(I),A0,A1,A2,A3,B0,B1,B2,B3,TU)
  655 CONTINUE
C
      IF (PHI(1).GE.0.0.AND.PHI(2).LT.0.0) GO TO 660
      IF (PHI(1).LT.0.0.AND.PHI(2).GE.0.0) GO TO 670
      IF (PHI(1).GE.0.0.AND.PHI(2).GE.0.0) GO TO 680
C
      NIND=0
      GO TO 760
C
  660 PACT(1)=PACTCR (PX(1),PY(1),PZ(1),A0,A1,A2,A3,B0,B1,B2,B3,TU)
      DO 661 I=1,NS
  661 P(I)=PACT(1)*P(I)
      CALL PTEAM (1,0.0,NEL)
      NIND=1
      GO TO 760
C
  670 PACT(2)=PACTCR (PX(2),PY(2),PZ(2),A0,A1,A2,A3,B0,B1,B2,B3,TU)
      DO 671 I=1,NS
  671 P(I)=PACT(2)*P(I)
      CALL PTEAM (2,0.0,NEL)
      NIND=2
      GO TO 760
C
  680 PACT(1)=PACTCR (PX(1),PY(1),PZ(1),A0,A1,A2,A3,B0,B1,B2,B3,TU)
      PACT(2)=PACTCR (PX(2),PY(2),PZ(2),A0,A1,A2,A3,B0,B1,B2,B3,TU)
      PPACT=PACT(1)
      IF (PACT(2).LT.PACT(1)) PPACT=PACT(2)
      DO 681 I=1,NS
  681 P(I)=PPACT*P(I)
      CALL PTEAM (3,0.0,NEL)
      NIND=3
C
  760 DO 761 I=1,NS
  761 F(I)=P(I)
      RETURN
      END


      SUBROUTINE PTEAM (NCODE,EFACT,NEL)
C
C     FORM NEW ELASTO-PLASTIC BEAM STIFFNESSES IN GLOBAL COORD. SYSTEM
C
      COMMON/EM/ MTYPE,LM(24),NO,NS,ASA(24,24),SA(12,24),S(12,12),PU,
     .           LPY,UMZ,TU,A0,A1,A2,A3,B0,B1,B2,B3,EPAR(12),LMS(12),
     .           EC(12),T(3,3),TTT(39),ASSA(24,24),SSA(12,24),DF(24),
     .           F(12),UP(12)
      COMMON/NEM/ EX(24),EX1(24),DEX(24),DFF(12),DP(12),FF(12),P(12),
     .            SS(12,12),UEXJ(12)
      DIMENSION YS(12,2),BS(12,2),AA(2,2),BB(2,2),UU(12),CPC(2)
C
      DO 10 J=1,2
      DO 10 I=1,12
      YS(I,J)=0.
   10 BS(I,J)=0.
      DO 20 J=1,2
      DO 20 I=1,2
   20 AA(I,J)=0.
C
      NC=1
      IF (NCODE.EQ.3) NC=2
      K=0
      SIGN=-1.0
      IF (NCODE.NE.2) GO TO 30
      K=6
      SIGN= 1.0
   30 DO 100 I=1,NC
      FX=P(K+1)*SIGN/PU
      FY=P(K+5)/UMY
      FZ=P(K+6)/UMZ
      CALL GRAUN (FX,FY,FZ,GFX,GFY,GFZ,A0,A1,A2,A3,B0,B1,B2,B3,TU)
      YS(K+1,I)=GFX*SIGN/PU
      YS(K+5,I)=GFY/UMY
      YS(K+6,I)=GFZ/UMZ
      K=K+6
      SIGN=-1.0*SIGN
  100 CONTINUE
C
      DO 200 K=1,NC
      DO 200 I=1,12
      XX=0.
      DO 125 J=1,12
      XX=XX+S(I,J)*YS(J,K)
  125 CONTINUE
  200 BS(I,K)=XX
C
      DO 300 I=1,NC
      DO 300 J=1,NC
      XX=0.
```

```
      DO 250 K=1,12
      XX=XX+YS(K,I)*BS(K,J)
  250 CONTINUE
  300 BB(I,J)=XX
C
      DET=BB(1,1)
      IF (NCDE.NE.3) GO TO 310
      DET=BB(1,1)*BB(2,2)-BB(1,2)*BB(2,1)
  310 IF (DET.LT.1.0E-30) GO TO 320
      AA(1,1)=1.0/DET
      IF (NCDE.NE.3) GO TO 320
      AA(1,1)= BB(2,2)/DET
      AA(1,2)=-BB(1,2)/DET
      AA(2,1)= AA(1,2)
      AA(2,2)= BB(1,1)/DET
C
  320 DO 400 I=1,12
      DO 380 J=1,12
      XX=0.
      DO 350 K=1,NC
      DO 350 L=1,NC
      XX=XX+BS(I,K)*AA(K,L)*BS(J,L)
  350 CONTINUE
  380 SS(I,J)=S(I,J)-XX
      IF (SS(I,I).LT.0.1*S(I,I)) SS(I,I)=0.1*S(I,I)
  400 CONTINUE
C
C     TRANSFORM TO GLOBAL COORD. SYSTEM
C
      DO 410 I=1,288
  410 SSA(I)=0.
      DO 450 LA=1,10,3
      LB=LA+2
      DO 450 MA=1,10,3
      MB=MA-1
      DO 450 I=LA,LB
      DO 450 JM=1,3
      J=JM+MB
      XX=0.
      DO 440 KK=1,3
      XX=XX+SS(I,KK+MB)*T(KK,JM)
  440 CONTINUE
  450 SSA(I,J)=XX
C
      DO 452 I=1,576
  452 ASSA(I)=0.
      DO 460 LA=1,10,3
      LB=LA-1
      DO 460 MA=1,10,3
      MB=MA+2
      DO 460 IL=1,3
      I=IL+LB
      DO 460 J=MA,MB
      XX=0.
      DO 455 KK=1,3
      XX=XX+T(KK,IL)*SSA(KK+LB,J)
  455 CONTINUE
  460 ASSA(I,J)=XX
C
C     TRANSFORM TO MASTER DEGREES OF FREEDOM
C
      DO 461 I=1,12
  461 UU(I)=DEX(I)
      KK=0
      KD=12
      DO 500 NF=1,12,6
      DO 490 K=1,3
      I=K+NF-1
      IF (LPS(I).LE.0) GO TO 490
      KK=KK+1
      D1=EC(KK)
      KK=KK+1
      D2=EC(KK)
      DO 470 II=1,KC
      ASSA(KD+1,II)=ASSA(I,II)*D1
      ASSA(KD+2,II)=ASSA(I,II)*D2
      ASSA(II,KD+1)=ASSA(II,I)*D1
      ASSA(II,KD+2)=ASSA(II,I)*D2
  470 CONTINUE
      DO 480 II=1,NS
      SSA(II,KD+1)=SSA(II,I)*D1
      SSA(II,KD+2)=SSA(II,I)*D2
  480 CONTINUE
      ASSA(KD+1,KD+1)=ASSA(I,I)*D1**2
      ASSA(KD+2,KD+2)=ASSA(I,I)*D2**2
      ASSA(KD+1,KD+2)=ASSA(I,I)*D1*D2
      ASSA(KD+2,KD+1)=ASSA(KD+1,KD+2)
      UU(I)=DEX(I)+DEX(KD+1)*D1+DEX(KD+2)*D2
      KD=KD+2
  490 CONTINUE
  500 CONTINUE
C
C     COMPUTE ELEMENT PLASTIC DEFORMATIONS
C
      IF (EFACT.EQ.0.0) GO TO 600
      IF (DET.LT.1.0E-30) GO TO 600
      DO 540 LA=1,10,3
      LB=LA+2
      MB=LA-1
      DO 540 I=LA,LB
      II=I-MB
      XX=0.
      DO 530 K=1,3
      XX=XX+T(K,II)*UU(K+MB)
  530 CONTINUE
  540 DEXJ(I)=XX
C
      DO 550 K=1,NC
      XX=0.
      DO 545 L=1,NC
      DO 545 J=1,12
  545 XX=XX+AA(K,L)*BS(J,L)*DEXJ(J)*EFACT
  550 DPD(K)=ABS(XX)
C
      DO 560 I=1,12
      XX=0.
      DO 555 K=1,NC
  555 XX=XX+YS(I,K)*DPD(K)
  560 UP(I)=UP(I)+XX
C
  600 RETURN
      END
```

```
      FUNCTION YIELD (PX,PY,PZ,A0,A1,A2,A3,B0,B1,B2,B3,TU)
C
C     DETERMINE VALUE OF YIELD FUNCTION
C
      IF (PX.LT.-0.9) GO TO 100
      IF (PX.GT.TU-0.1) GO TO 100
C
      PXY=A0+A1*PX+A2*PX**2+A3*PX**3
      PXZ=B0+B1*PX+B2*PX**2+B3*PX**3
      YIELD=(PY/PXY)**2+(PZ/PXZ)**2-1.0
      RETURN
C
  100 YIELD=1.0
      RETURN
      END


      FUNCTION FACTOR (TX,TY,TZ,DTX,DTY,DTZ,A0,A1,A2,A3,B0,B1,B2,B3,TU)
C
C     DETERMINE OVERSHOOT FACTOR OF STRESS INCREMENT TO YIELD SURFACE
C
      C=0.0
      IF (ABS(DTX).LT.1.0E-30) GO TO 5
      IF (TX.LE.-1.0) C=(TX+1.0)/DTX
      IF (TX.GE. TU ) C=(TX-TU )/DTX
    5 FX=TX-C*DTX
      FY=TY-C*DTY
      FZ=TZ-C*DTZ
      DFX=(1.0-C)*DTX
      DFY=(1.0-C)*DTY
      DFZ=(1.0-C)*DTZ
      FXY=A0+A1*FX+A2*FX**2+A3*FX**3
      FXZ=B0+B1*FX+B2*FX**2+B3*FX**3
C
      N=0
      ACTOR=1.0
      YC=FXY
      Y1=A1*DFX+2.0*A2*FX*DFX+3.0*A3*FX*FX*DFX
      Y2=A2*DFX*DFX+3.0*A3*FX*DFX*DFX
      Y3=A3*DFX*DFX*DFX
      ZC=FXZ
      Z1=B1*DFX+2.0*B2*FX*DFX+3.0*B3*FX*FX*DFX
      Z2=B2*DFX*DFX+3.0*B3*FX*DFX*DFX
      Z3=B3*DFX*DFX*DFX
C
   10 N=N+1
      YY =YC-Y1*ACTOR+Y2*ACTOR**2-Y3*ACTOR**3
      YY1=FY-DFY*ACTOR
      YY2=-Y1+2.0*Y2*ACTOR-3.0*Y3*ACTOR**2
      ZZ =ZC-Z1*ACTOR+Z2*ACTOR**2-Z3*ACTOR**3
      ZZ1=FZ-DFZ*ACTOR
      ZZ2=-Z1+2.0*Z2*ACTOR-3.0*Z3*ACTOR**2
      IF (ABS(YY).LT.1.0E-30.OR.ABS(ZZ).LT.1.0E-30) GO TO 120
      P =(YY1/YY)**2+(ZZ1/ZZ)**2-1.0
C
      IF (ABS(P).LE.1.0E-10) GO TO 100
      IF (N.GT.100) GO TO 100
      DP=(2.0*YY1*(-DFY*YY-YY1*YY2)/(YY**3)) +
     .   (2.0*ZZ1*(-DFZ*ZZ-ZZ1*ZZ2)/(ZZ**3))
      IF (ABS(DP).LT.1.0E-30) GO TO 120
      ACTOR=ACTOR-P/DP
      GO TO 10
C
  100 FACTOR=ACTOR*(1.0-C)+C+1.0E-10
      IF (FACTOR.GT.1.0) FACTOR=1.0
      IF (FACTOR.LT.0.0) FACTOR=0.0
      RETURN
C
  120 ACTOR=1.0
      GO TO 100
      END


      FUNCTION PACTOR (TX,TY,TZ,A0,A1,A2,A3,B0,B1,B2,B3,TU)
C
C     DETERMINE REDUCTION FACTOR OF TOTAL STRESS VECTOR TO YIELD SURFACE
C
      TT=TU-0.1
      C =1.0
      IF (TX.LT.-0.9) C=-0.9/TX
      IF (TX.GT. TT ) C=  TT/TX
      FX=C*TX
      FY=C*TY
      FZ=C*TZ
      N=0
      ACTOR=1.0
C
   10 N=N+1
      PX=ACTOR*FX
      PY=ACTOR*FY
      PZ=ACTOR*FZ
      PXY=A0+A1*PX+A2*PX**2+A3*PX**3
      PXZ=B0+B1*PX+B2*PX**2+B3*PX**3
      IF (PXY.LT.1.0E-10.OR.PXZ.LT.1.0E-10) GO TO 200
      FP=(PY/PXY)**2+(PZ/PXZ)**2-1.0
      IF (ABS(FP).LE.1.0E-10) GO TO 100
      IF (N.GT.100) GO TO 200
      YY=A1*FX+2.0*A2*FX*PX+3.0*A3*FX*PX*PX**2
      ZZ=B1*FX+2.0*B2*FX*PX+3.0*B3*FX*PX*PX**2
      DP=2.0*PY*(FY*PXY-PY*YY)/PXY**3+2.0*PZ*(FZ*PXZ-PZ*ZZ)/PXZ**3
      ACTOR=ACTOR-FP/DP
      IF (ABS(ACTOR).GT.1.0E 10) GO TO 200
      GO TO 10
C
  100 PACTOR=ACTOR*C-1.0E-5
      IF (PACTOR.GT.1.0) PACTOR=1.0
      IF (PACTOR.LT.0.0) PACTOR=0.0
      RETURN
C
  200 N=0
      PMIN=0.
      PMAX=1.0
      IF (TX.LT.-1.0) PMAX=-1.0/TX
      IF (TX.GT. TU ) PMAX= 1.0/TX
C
  210 N=N+1
      P=(PMAX+PMIN)/2.0
      PX=P*TX
```

```
      PY=P*TY
      PZ=P*TZ
      IF (PX.GE.TU.OR.PX.LE.-1.0) GO TO 300
      PXY=A0+A1*PX+A2*PX**2+A3*PX**3
      PXZ=B0+B1*PX+B2*PX**2+B3*PX**3
      IF (PXY.LT.1.0E- 5.OR.PXZ.LT.1.0E- 5) GO TO 450
      FP=(PY/PXY)**2+(PZ/PXZ)**2-1.0
      GO TO 400
  300 FP=1.0
  400 IF (FP.GT.-1.0E-05.AND.FP.LT.0.) GO TO 500
      IF (N.GT.100) GO TO 450
      IF (FP) 410,500,420
  410 PMIN=P
      GO TO 210
  420 PMAX=P
      GO TO 210
C
  450 P=PMIN
  500 FACTOR=P
      RETURN
      END
```

```
      SUBROUTINE GRAON (FX,FY,FZ,GFX,GFY,GFZ,A0,A1,A2,A3,B0,B1,B2,B3,TU)
C
C     COMPLTE GRADIENT TO YIELD SURFACE
C
      IF (FX.GT.TU-0.1) GO TO 100
      FXY=A0+A1*FX+A2*FX**2+A3*FX**3
      FXZ=B0+B1*FX+B2*FX**2+B3*FX**3
      FYY=A1+2.0*A2*FX+3.0*A3*FX*FX
      FZZ=B1+2.0*B2*FX+3.0*B3*FX*FX
C
      GFX=(-2.0*FY*FY*FYY)/(FXY**3)+(-2.0*FZ*FZ*FZZ)/(FXZ**3)
      GFY=( 2.0*FY)/(FXY**2)
      GFZ=( 2.0*FZ)/(FXZ**2)
      RETURN
C
  100 GFX=1.0
      GFY=0.0
      GFZ=0.0
      RETURN
      END
```

```
      SUBROUTINE PBOUND (NEL,NINO)
C
C     CHECK YIELOING OF BOUNDARY ELEMENTS AND FORM BILINEAR STIFFNESSES
C
      COMMON/EM/ MTYPE,LM(24),ND,NS,ASA(24,24),SA(12,24),S(12,12),
     .           SY(6),EP(6),UE(6),PAR(6),T(3,3),TTT(63),ASSA(24,24),
     .           SSA(12,24),UF(24),F(12),UP(12)
      COMMON/NEM/ EX(24),EX1(24),DEX(24),OFF(12),OP(12),FF(12),P(12),
     .           SS(6,6),OEXJ(6),KY(6),FACTOR(6)
C
      DO 5 I=1,SC
    5 OP(I)=0.
      DO 10 I=1,NS
      XX=0.
      DO 8 J=1,NO
      XX=XX+SA(I,J)*OEX(J)
    8 CONTINUE
      OFF(I)=XX
      FF(I)=F(I)+OFF(I)
   10 P(I)=F(I)
C
C     CHECK YIELDING OF BOUNDARY ELEMENTS
C
      DO 20 I=1,NS
      KY(I)=0
      FACTOR(I)=0.
      AA=ABS(OFF(I))
      XX=SY(I)+EP(I)*UP(I)
      YY=SY(I)-EP(I)*UP(I)
      IF (FF(I).LE.XX) GO TO 15
      KY(I)=1
      IF (AA.LT.1.0E-10) GO TO 20
      FACTOR(I)=(FF(I)-XX)/OFF(I)
      GO TO 20
   15 IF (FF(I).GE.-YY) GO TO 20
      KY(I)=-1
      IF (AA.LT.1.0E-10) GO TO 20
      FACTOR(I)=(FF(I)+YY)/OFF(I)
   20 CONTINUE
C
      DO 30 I=1,NS
      IF (KY(I).NE.0) GO TO 100
   30 CONTINUE
C
C     A. ELEMENT IN ELASTIC STATE
C
      DO 50 I=1,NS
   50 F(I)=FF(I)
      NINO=0
      GO TO 500
C
C     B. ELEMENT IN PLASTIC STATE
C
C     B1. COMPUTE ELEMENT STRESSES IN PLASTIC STATE
C
  100 DO 120 LA=1,6,3
      LB=LA+2
      MB=LA-1
      DO 120 I=LA,LB
      II=I-MB
      XX=0.
```

```
      DO 115 K=1,3
  115 XX=XX+T(II,K)*DEX(K+MB)
  120 DEXJ(I)=XX
C
      DO 132 I=1,6
      DO 132 J=1,6
  132 SS(I,J)=S(I,J)
      DO 140 I=1,6
      IF (KY(I)) 135,140,135
  135 SS(I,I)=EP(I)
  140 CONTINUE
C
      DO 150 I=1,NS
      P(I)=P(I)+(1.0-FACTOR(I))*OFF(I)+FACTOR(I)*SS(I,I)*DEXJ(I)
      F(I)=P(I)
      UP(I)=UP(I)+FACTOR(I)*DEXJ(I)
  150 CONTINUE
C
C     B2. FORM BILINEAR STIFFNESSES IN GLOBAL COORD. SYSTEM
C
      DO 180 I=1,288
  180 SSA(I)=0.
      DO 200 LA=1,6,3
      LB=LA+2
      DO 200 MA=1,6,3
      MB=MA-1
      DO 200 I=LA,LB
      DO 200 JM=1,3
      J=JM+MB
      XX=0.
      DO 190 K=1,3
      YY=SS(I,K+MB)
      IF (YY.EQ.0.) GO TO 190
      XX=XX+YY*T(K,JM)
  190 CONTINUE
  200 SA(I,J)=XX
C
      DO 210 I=1,576
  210 ASSA(I)=0.
      DO 250 LA=1,6,3
      LB=LA-1
      DO 250 MA=1,6,3
      MB=MA+2
      DO 250 IL=1,3
      I=IL+MB
      DO 250 J=MA,MB
      XX=0.
      DO 240 K=1,3
      XX=XX+T(K,IL)*SSA(K+LB,J)
  240 CONTINUE
  250 ASSA(I,J)=XX
      NIND=1
C
  500 DO 600 I=1,NC
  600 DF(I)=0.
      RETURN
      END
```

```
      SUBROUTINE NEXPJB (NEL,NIND)
C
C     CHECK JOINT CONDITIONS AND FORM NEW JOINT STIFFNESSES IF NEEDED
C
      COMMON/EM/ MTYPE,LM(24),ND,NS,ASA(24,24),SA(12,24),S(12,12),
     .           CCF,GAP,TG,STIE,SYTIE,NTIE,XTIE(6),TRACE,COULTIF,
     .           SJ,KD(6),A(6,6),T(3,3),PTIE,KKD(6),TTT(20),SSSA(24,24),
     .           SSA(12,24),DF(24),F(6),FT(6),U(6),UPTIE(6)
      COMMON/NEM/ EX(24),EXI(24),DEX(24),DEF(12),DP(12),EXJ(12),EXJI(12)
     .           ,SS(12,12),UI(6),TK(6)
      COMMON/COLLOMB/ CF(20,6)
      DIMENSION CFF(6)
C
      DO 10 I=1,12
   10 DP(I)=0.
      DO 12 I=1,ND
   12 DF(I)=0.
C
C     COMPUTE LOCAL JOINT RELATIVE DISPLACEMENTS AND VELOCITIES
C
      DO 20 LA=1,12,6
      LB=LA+5
      MB=LA-1
      DO 20 I=LA,LB
      II=I-MB
      XX=0.
      YY=0.
      DO 15 KK=1,6
      K=KK+MB
      XX=XX+A(II,KK)*EX(K)
      YY=YY+A(II,KK)*EXI(K)
   15 CONTINUE
      EXJ (I)=XX
      EXJI(I)=YY
   20 CONTINUE
      DO 50 I=1,6
      U (I)=EXJ (I+6)-EXJ (I)
      UI(I)=EXJI(I+6)-EXJI(I)
   50 CONTINUE
C
C     CHECK CONNECTIVITY OF THE EXPANSION JOINT
C
      PTIE=1000.
      IF (NTIE.EQ.0) GO TO 61
      DO 60 I=1,NTIE
      IF (PTIE.GT.UPTIE(I)) PTIE=UPTIE(I)
   60 CONTINUE
   61 PTIE=PTIE+TG
      DO 65 I=1,6
   65 KKD(I)=KD(I)
C
      IF (U(1).LT.-CAP.AND.UI(1).LT.0.0) KKD(1)=-1
      IF (U(1).GT.PTIE) KKD(1)= 1
C
      IF (U(4).LT.-CAP.AND.UI(4).LT.0.0) KKD(4)=-1
      IF (U(4).GT.PTIE) KKD(4)= 1
C
      DO 70 I=1,6
      IF (KKD(I).NE.KU(I)) GO TO 100
   70 CONTINUE
C
```

```
      DO 80 I=1,6
      XX=0.
      DO 75 J=1,NO
      XX=XX+SA(I,J)*DEX(J)
   75 CONTINUE
   80 DFF(I)=XX
      NINO=0
      GO TO 120
C
C     FORM NEW EXPANSION JOINT CONDITIONS
C
  100 CALL NXPJB
      DO 110 I=1,6
      XX=0.
      DO 105 J=1,NO
      XX=XX+SSA(I,J)*DEX(J)
  105 CONTINUE
  110 DFF(I)=XX
      NINO=1
C
  120 DO 130 I=1,6,3
      IF (DFF(I).EQ.0.0) F(I)=0.0
  130 CONTINUE
      DO 150 I=1,6
  150 F(I)=F(I)+DFF(I)
C
C     COMPUTE COULOMB FRICTIONAL FORCES
C
      IF (SJ*F(3).LE.0.0.AND.SJ*F(6).LE.0.0) RETURN
      DO 200 I=1,6
  200 CFF(I)=0.
      DO 250 I=1,6,3
      II=I+2
      IF (SJ*F(II).LE.0.0) GO TO 250
  215 IF (U1(I)) 225,220,225
  220 CFF(I)=-CCF*SJ*F(II)
      GO TO 240
  225 CFF(I)= CCF*SJ*F(II)
  240 DCF=CFF(I)-CF(NEL,I)
      DP(I)=DP(I)+DCF
      DP(I+6)=-DP(I)
      CF(NEL,I)=CFF(I)
  250 CONTINUE
C
C     TRANSFORM FRICTIONAL FORCE INCREMENTS TO GOLBAL COORD. SYSTEM
C
      DO 300 LA=1,12,6
      LB=LA-1
      DO 300 IL=1,6
      I=IL+LB
      XX=0.
      DO 255 K=1,6
      XX=XX+A(K,IL)*DP(K+LB)
  255 CONTINUE
  300 DF(I)=XX
      RETURN
      END



      SUBROUTINE NXPJB
C
C     FORM NEW EXPANSION JOINT STIFFNESSES
C
      COMMON/EM/ MTYPE,LM(24),NO,NS,ASA(24,24),SA(12,24),S(12,12),
     *           CCF,GAP,TG,STIE,SYTIE,NTIE,XTIE(8),TRACE,D,UETIE,
     *           SJ,KO(6),A(6,6),T(3,3),PTIE,KKD(6),TTT(20),ASSA(24,24),
     *           SSA(12,24),DF(24),F(6),FT(6),U(6),UPTIE(6)
      COMMON/NEW/ EEEE(72),DFF(12),DP(12),EXJ(12),EXJ1(12),SS(12,12),
     *           U1(6),TK(6)
C
C     COMPUTE TIE STIFFNESSES
C
      DO 5 I=1,6
    5 TK(I)=0.
      IF (NTIE.EQ.0) GO TO 70
C
      DO 12 I=1,NTIE
      XX=(U(1)+U(4))/2.0+XTIE(I)*(U(1)-U(4))/D
      XX=XX-TG
      IF (XX.LT.UPTIE(I)) GO TO 10
      IF (XX-UPTIE(I)-UETIE) 9,9,8
    8 UPTIE(I)=XX-UETIE
      TK(I)=0.05
      FT(I)=SYTIE
      GO TO 12
    9 TK(I)=1.0
      FT(I)=STIE*(XX-UPTIE(I))
      GO TO 12
   10 FT(I)=0.
   12 CONTINUE
C
      TAA=0.
      TAB=0.
      TBB=0.
      DO 20 I=1,NTIE
      TAA=TAA+TK(I)*STIE*(D/2.0+XTIE(I))**2/D**2
      TAB=TAB+TK(I)*STIE*(D/2.0+XTIE(I))*(D/2.0-XTIE(I))/D**2
      TBB=TBB+TK(I)*STIE*(D/2.0-XTIE(I))**2/D**2
   20 CONTINUE
      GO TO 80
C
   70 TAA=0.
      TAB=0.
      TBB=0.
C
C     FORM NEW LOCAL JOINT STIFFNESSES
C
   80 DO 100 I=1,144
  100 SS(I)=0.
C
      IF (KKD(1)) 110,120,115
  110 SS(1,1)=TRACE
      GO TO 120
  115 SS(1,1)=TAA
      SS(1,4)=TAB
      SS(4,1)=TAB
      SS(4,4)=TBB
C
  120 IF (KKD(2)) 140,140,130
  130 SS(2,2)=TRACE
C
  140 IF (KKD(3)) 160,160,150
```

```
  150 SS(3,3)=TRACE
C
  160 IF (KKD(4)) 170,180,175
  170 SS(4,4)=TRACE
      GO TO 180
  175 SS(1,1)=TAA
      SS(1,4)=TAB
      SS(4,1)=TAB
      SS(4,4)=TBB
C
  180 IF (KKD(5)) 200,200,190
  190 SS(5,5)=TRACE
C
  200 IF (KKD(6)) 220,220,210
  210 SS(6,6)=TRACE
C
  220 DO 250 LA=1,12,6
      LB=LA+5
      DO 250 MA=1,12,6
      MB=MA+5
      IF (LA.EQ.1.AND.MA.EQ.1) GO TO 250
      SIGN=-1.0
      IF (LA.EQ.7.AND.MA.EQ.7) SIGN= 1.0
      DO 240 I=LA,LB
      II=I-LA+1
      DO 240 J=MA,MB
      JJ=J-MA+1
      SS(I,J)=SIGN*SS(II,JJ)
  240 CONTINUE
  250 CONTINUE
C
C     TRANSFORM TO GLOBAL COORD. SYSTEM
C
      DO 310 I=1,288
  310 SSA(I)=0.
      DO 330 LA=1,12,6
      LB=LA+5
      DO 330 MA=1,12,6
      MB=MA-1
      DO 330 I=LA,LB
      DO 330 JM=1,6
      J=JM+MB
      XX=0.
      DO 325 K=1,6
      YY=SS(I,K+MA)
      IF (YY.EQ.0.) GO TO 325
      XX=XX+YY*A(K,JM)
  325 CONTINUE
  330 SSA(I,J)=XX
C
      DO 351 I=1,576
  351 ASSA(I)=0.
      DO 360 LA=1,12,6
      LB=LA-1
      DO 360 MA=1,12,6
      MB=MA+5
      DO 360 IL=1,6
      I=IL+LB
      DO 360 J=MA,MB
      XX=0.
      DO 355 K=1,6
      XX=XX+A(K,IL)*SSA(K+LB,J)
  355 CONTINUE
  360 ASSA(I,J)=XX
      RETURN
      END
```


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